APPENDIX A
HYDRAULIC ANALYSIS

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1.0 INTRODUCTION

1.1 Study Authority

This hydraulic design appendix describes the technical aspects of the Port Lions Navigation Improvements project. It provides the background for determining the Federal interest in the major construction features including breakwaters and operation and maintenance.

1.2 Scope of Study

The State of Alaska Department of Transportation and Public Facilities (ADOT&PF) acting as the local sponsor for the city of Port Lions requested the Corps of Engineers to conduct a feasibility study of navigation improvements. Additional wave protection for vessel moorage was identified as a critical issue facing the community. The following objective was identified to accomplish navigation improvements at Port Lions prior to initiating the engineering analysis:

- Prevent reoccurring storm waves from damaging the float system and vessels in the existing harbor by providing a fully protected mooring area for the fleet.
- The project purpose is to provide for a safe and efficient harbor in an economically and environmentally sound manner that satisfies the above objective.

2.0 CLIMATOLOGY, METEOROLOGY, COASTAL HYDRAULICS

2.1 Climatology

Port Lions is located on Kodiak Island, approximately 30 air-miles northwest of the city of Kodiak and 260 air-miles southwest of Anchorage. Port Lions and the contiguous marine waters of Settler Cove are at latitude 57°53' N and longitude 152°53' W as shown on Figure 1 in the main report. The cove opens to Kizuyak Bay and Marmot Bay toward the northeast. The existing harbor in Settler Bay lies to the northeast of the city of Port Lions. The area has a maritime climate primarily influenced by strong low-pressure centers generated in the Gulf of Alaska and North Pacific Ocean. Cool summers, mild winters, and year-round rainfall characterize the climate. Snow falls primarily between November and April and the average annual snowfall is 75 inches. Rains may occur any time of the year, and annual average precipitation per year is 54 inches. The wettest months occur in the fall with October and November having the highest monthly and record rainfall. Fog is generally common and occurs under certain conditions during the summer months. Normal winter temperature ranges from 10 °F to 40 °F, while summer temperatures range from 55 °F to 70 °F. Temperatures can reach record lows of –5 °F and record highs of +80 °F.

2.2 Wind Data

Predominant winds at Port Lions are generally caused by low-pressure systems that track in an easterly direction across the North Pacific Ocean and Gulf of Alaska. Strong winds occur throughout the year; however, wind patterns have a seasonal component. Summer winds are generally from the east and are lighter. Winter winds are predominantly from the northwest and are generally stronger. Historical wind speed and direction data (period of record 1973 to 1997) for the Kodiak airport is summarized in the wind roses shown in figures 1 through 13. The Port Lions area as with most of Kodiak Island is known for intense storms that occur from various directions. According to local residents, the severe and damage-causing storms usually occur in the fall and winter and come from the northeast direction. These storms are relatively infrequent, however, they can occur two to three times a year according to local residents. High winds and waves have caused severe damage to the float system in the existing harbor in Settler Cove under such conditions. Local residents have estimated wind speeds to be a sustained 65 to 80 miles per hour (mph) during major storms. Gusts of up to 100 mph have been observed.

A wind data summary was presented by the Corps of Engineers in the June 1977 Detailed Project Report for Port Lions. Wind data from an onsite onshore anemometer for a period of record of 1970 to 1975 was analyzed. The resulting estimate for the 50-year wind speed of 40 miles per hour (mph) was determined.

An additional wind data analysis was prepared by the Corps of Engineers for the June 1982 Letter Report for Port Lions following the failure of the armor stone layer on the newly constructed breakwater. Analyses of several types of data were used to revise the original estimate of the 50-year design wind to be used in the design wave determination for breakwater repair. These data included National Weather Service data for the airport at the city of Kodiak, recorded wind velocities from an anemometer and wave-rider buoy at

Kodiak, an analysis of local winds published by H.C.S. Thom, an evaluation of observed wind velocities at Port Lions during the November 1981 storm, and a wind hindcast study conducted by Waterways Experiment Station (WES) in May of 1982. The resulting 50-year design wind determined is summarized in the following table.

	Alaska District Frequency Analysis		WES Hindcast Analysis	
	JONSWAP	SMB	JONSWAP	SMB
Design wind speed (mph)	66	55	89	56
Duration (hours)	4.6	1.7	3.8	1.7

The terms JONSWAP and SMB are used to distinguish winds estimated with differing adjustments. The JONSWAP winds indicated in the above table included adjustments for height of the anemometer, drag coefficient, and air-sea temperature difference. The SMB winds included adjustments for height of the anemometer only.

The storm that produced damage causing wave conditions shortly after initial breakwater construction at Port Lions occurred November 9 thru 12, 1981. This storm system was part of a major low-pressure center that moved through the North Pacific Ocean during that week. Local television stations in Anchorage indicated that it would have been classified as a hurricane had it been on the east coast. The National Weather Service at the Kodiak Airport recorded the following wind gusts for the period of November 9 thru 12, 1981:

Date	Direction	Gust Speed (mph)
November 9, 1981	NE	63
November 10, 1981	SE	43
November 11, 1981	NE	48
November 12, 1981	NE	48

The winds on November 9 were described as gusty. The winds on November 11 and 12 were sustained near the peak levels for most of both days. Local residents indicated that wind velocities in Port Lions were between 35 and 45 mph on a sustained basis. The estimated wind velocity at the harbor was approximately 60 mph during this storm event. Long time residents of the Port Lions area characterize winds of this magnitude as not unusual. Local accounts of larger storms include northeasterly winds of 80 mph in January of 1976 and 80 to 100 mph in November of 1980, both storms occurring prior to construction of the original breakwater.

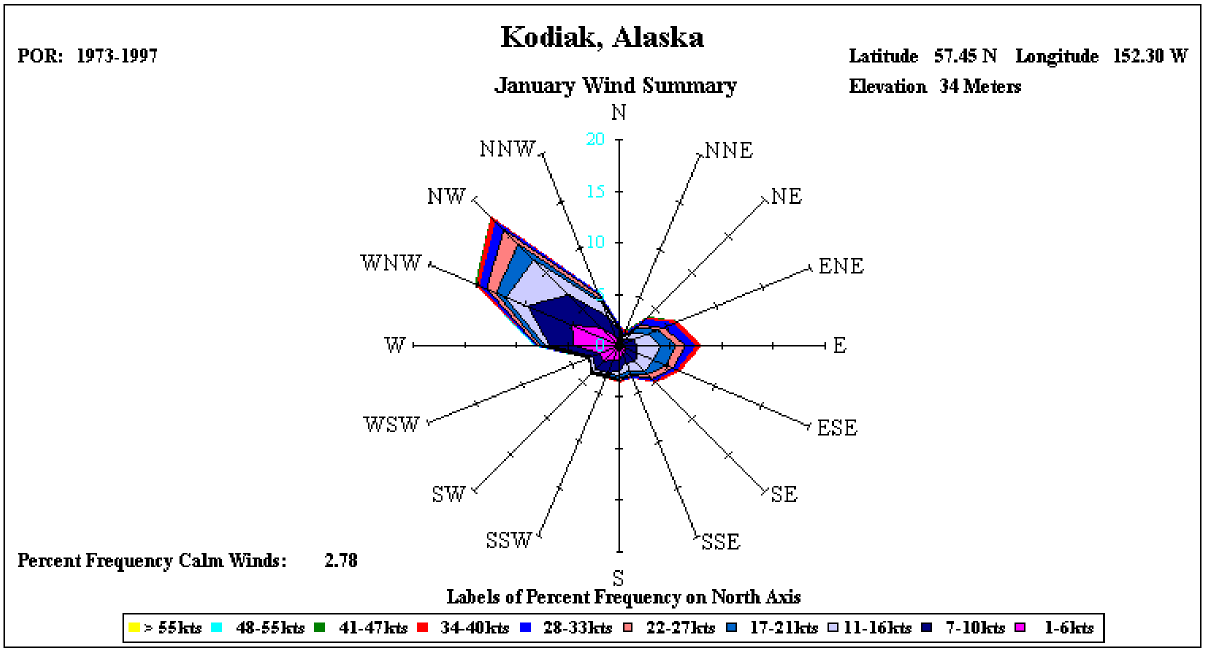


Figure 1. Wind Rose

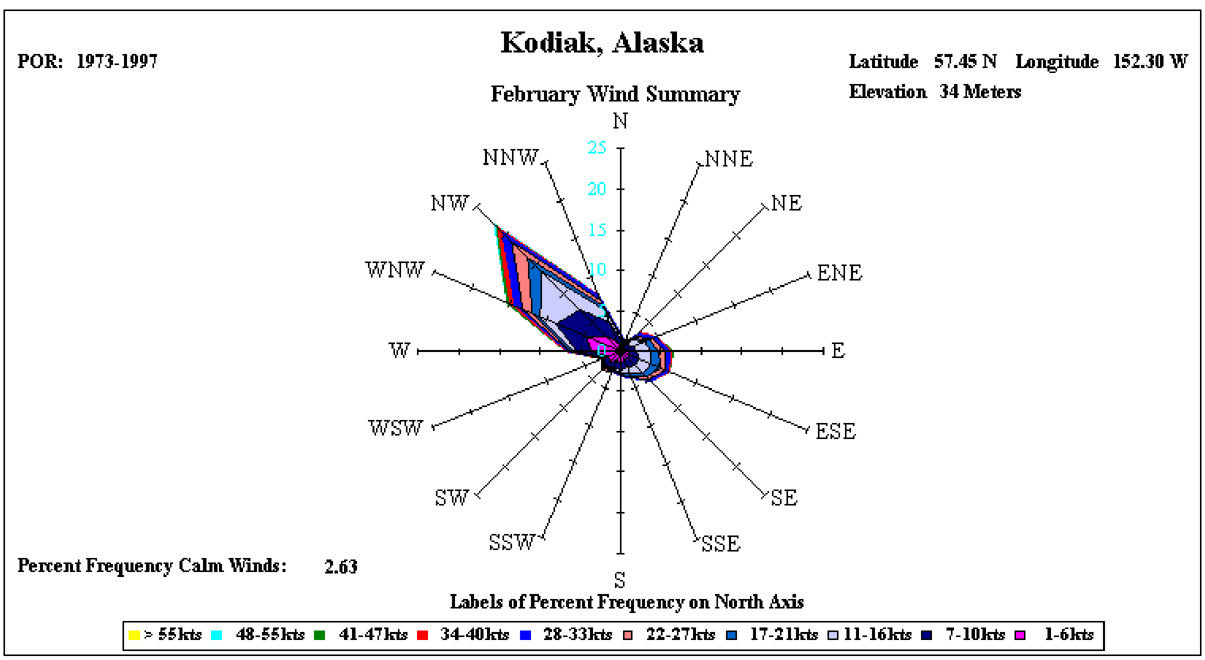
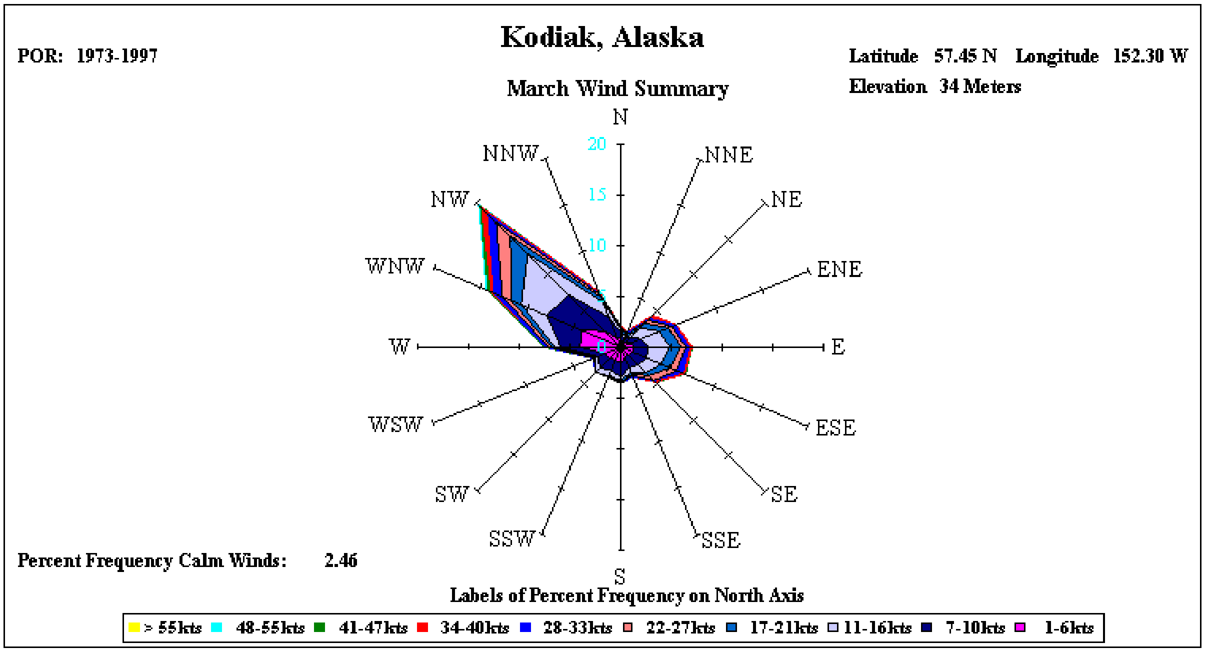


Figure 2. Wind Rose



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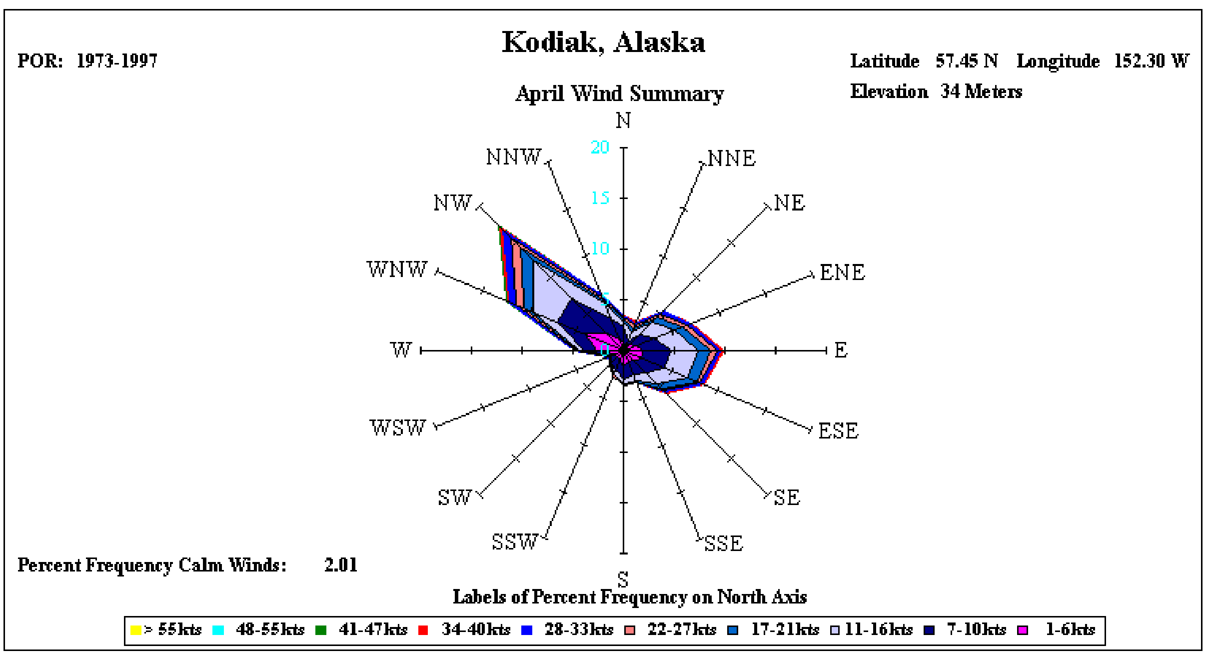

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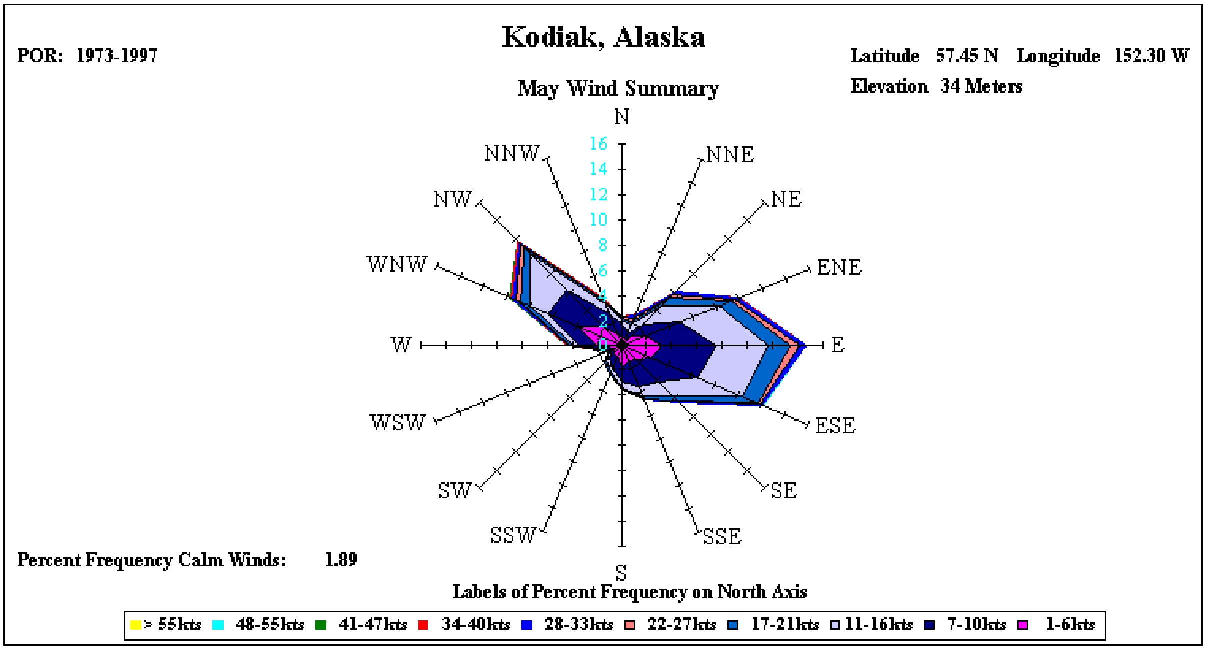

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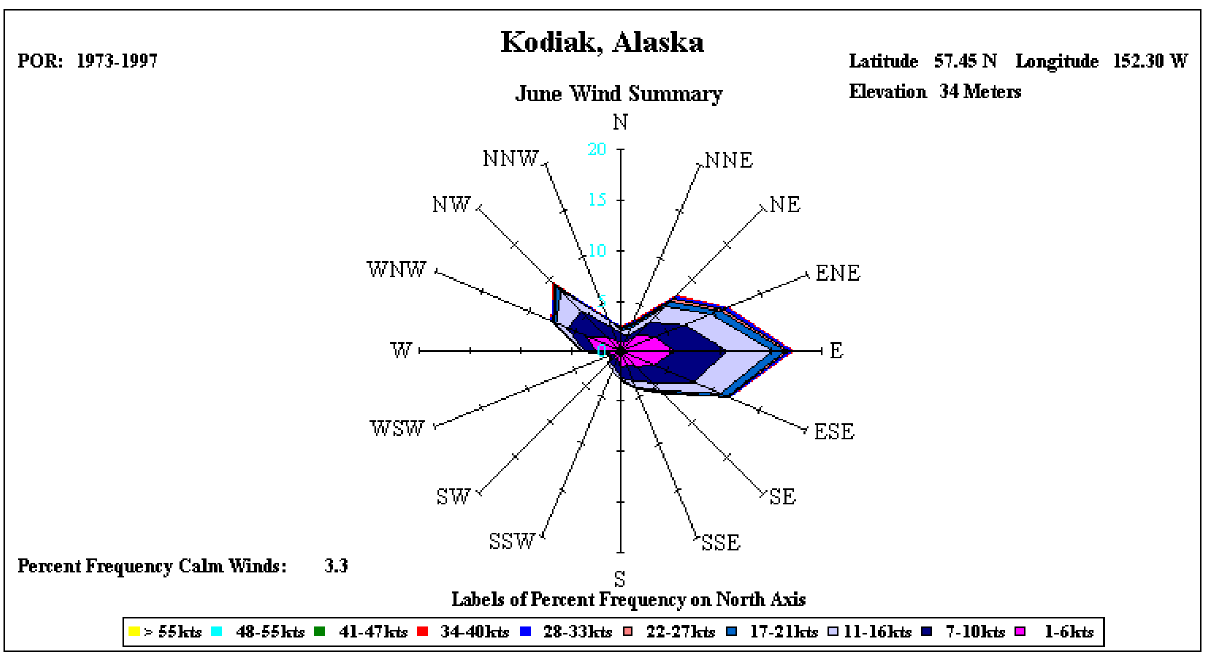

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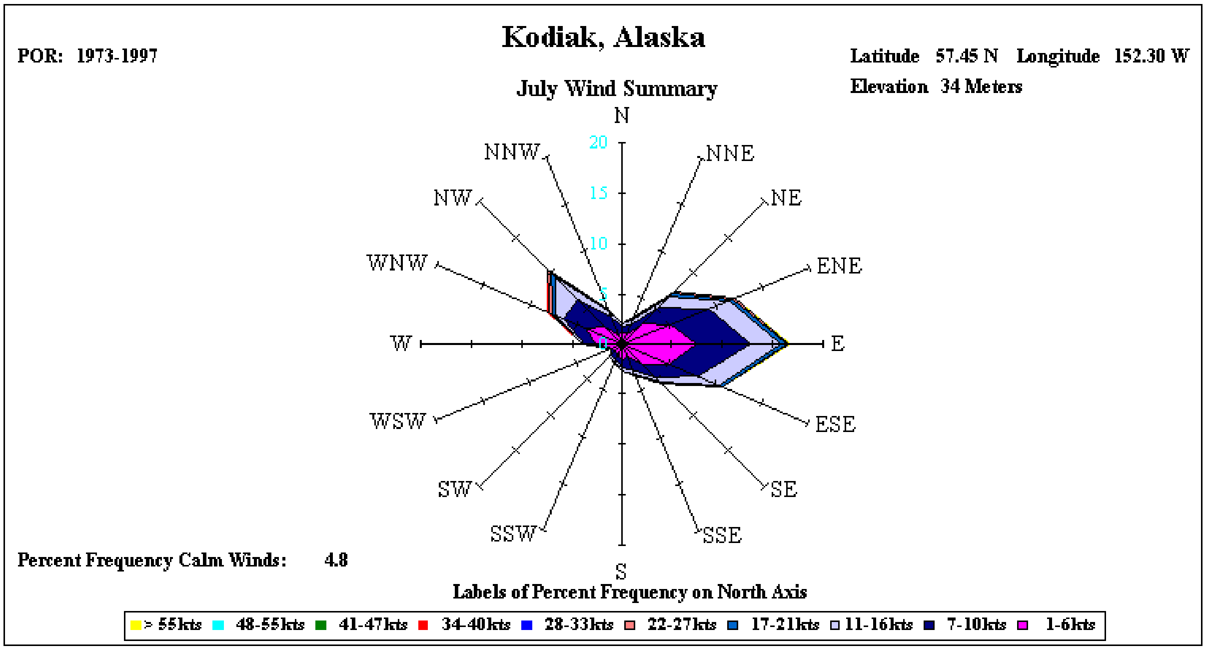

Figure 7. Wind Rose

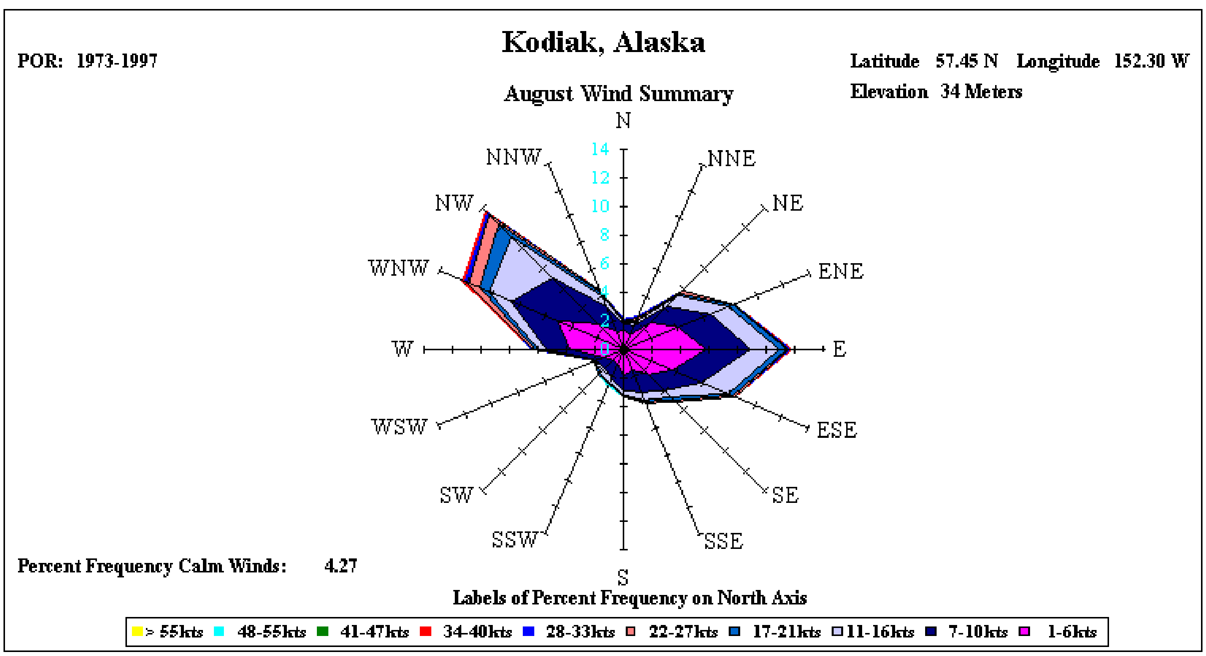

Figure 8. Wind Rose

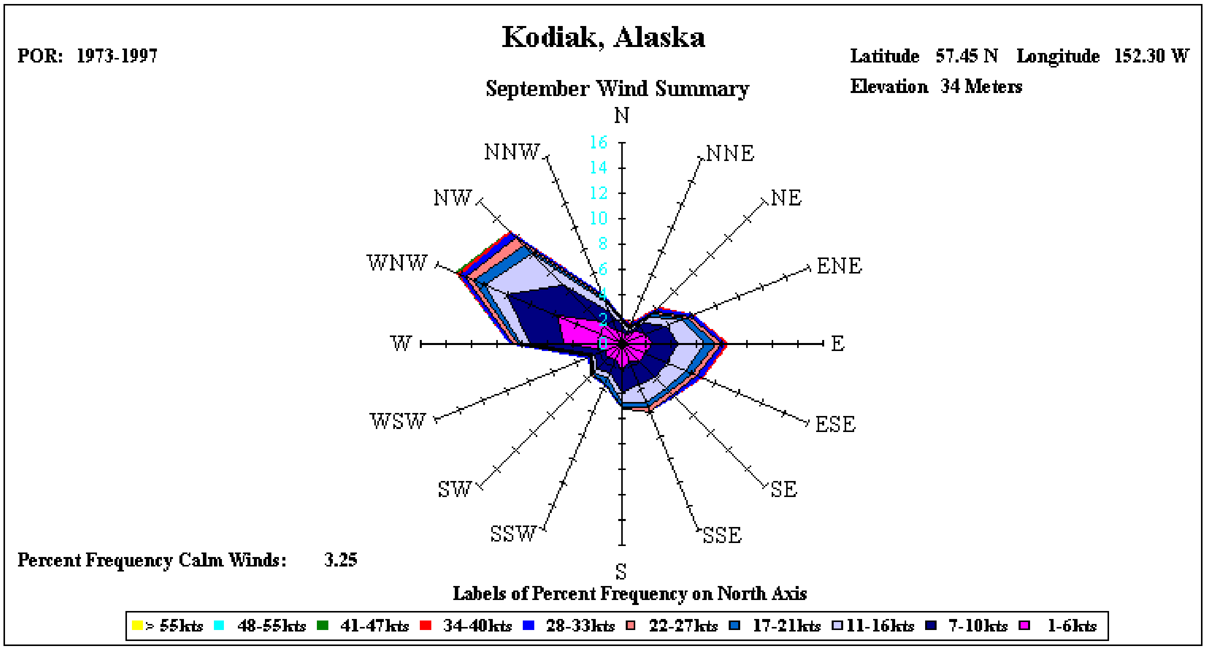

Figure 9. Wind Rose

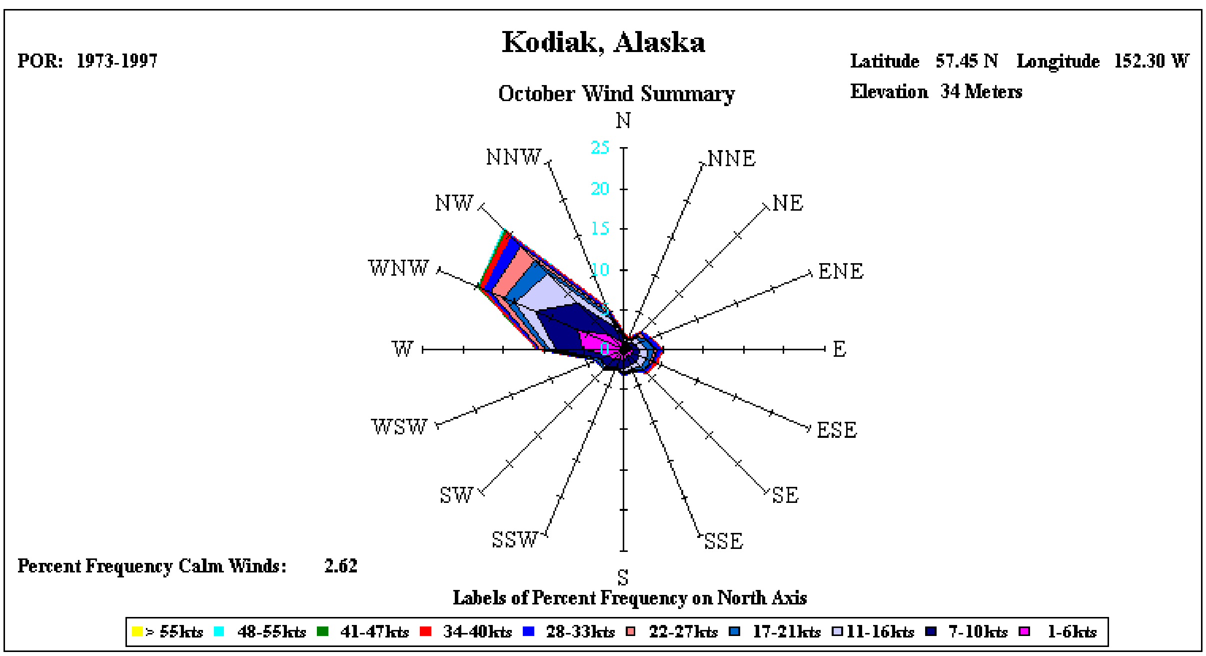

Figure 10. Wind Rose

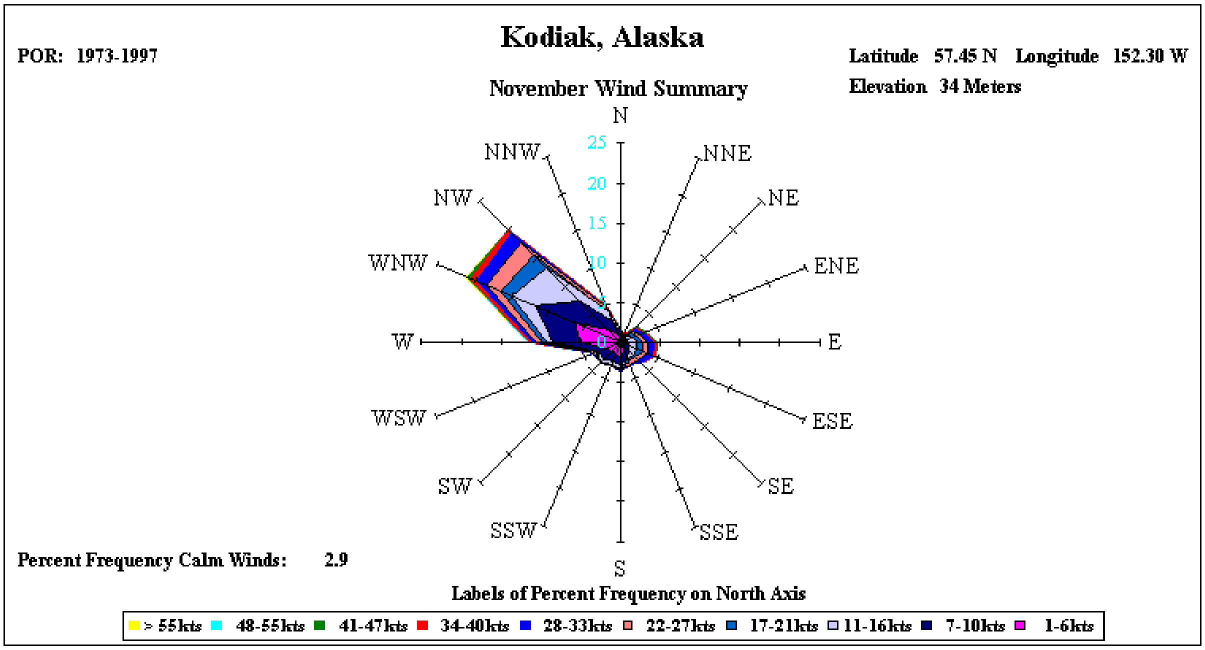

Figure 11. Wind Rose

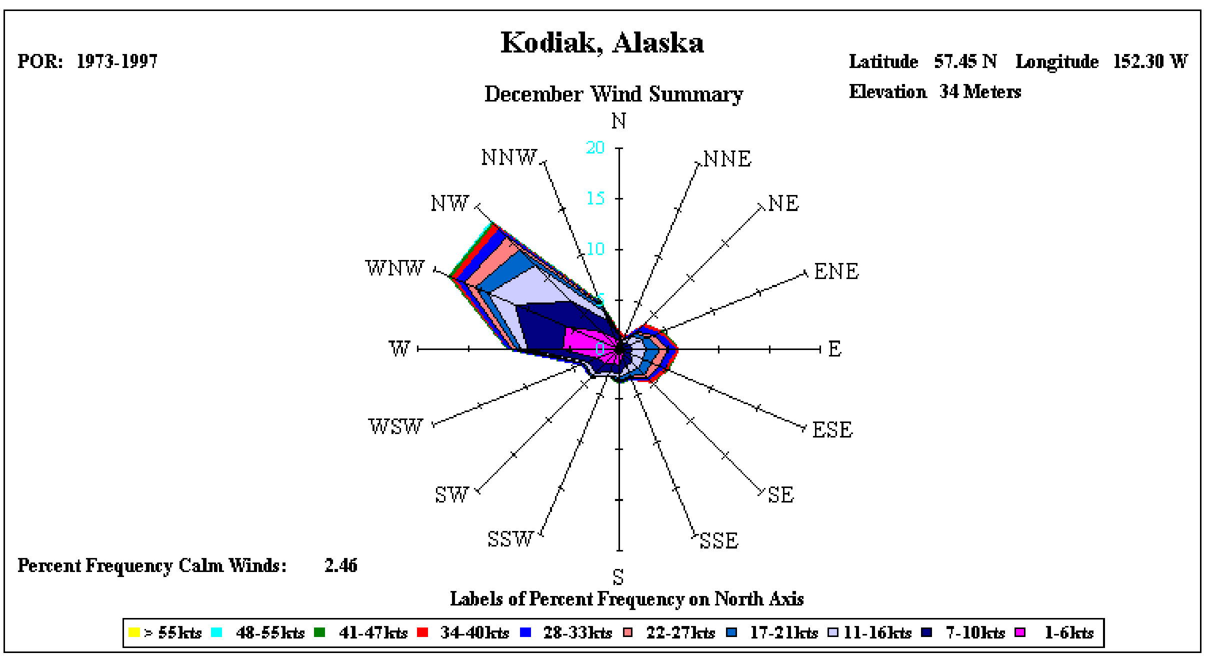

Figure 12. Wind Rose

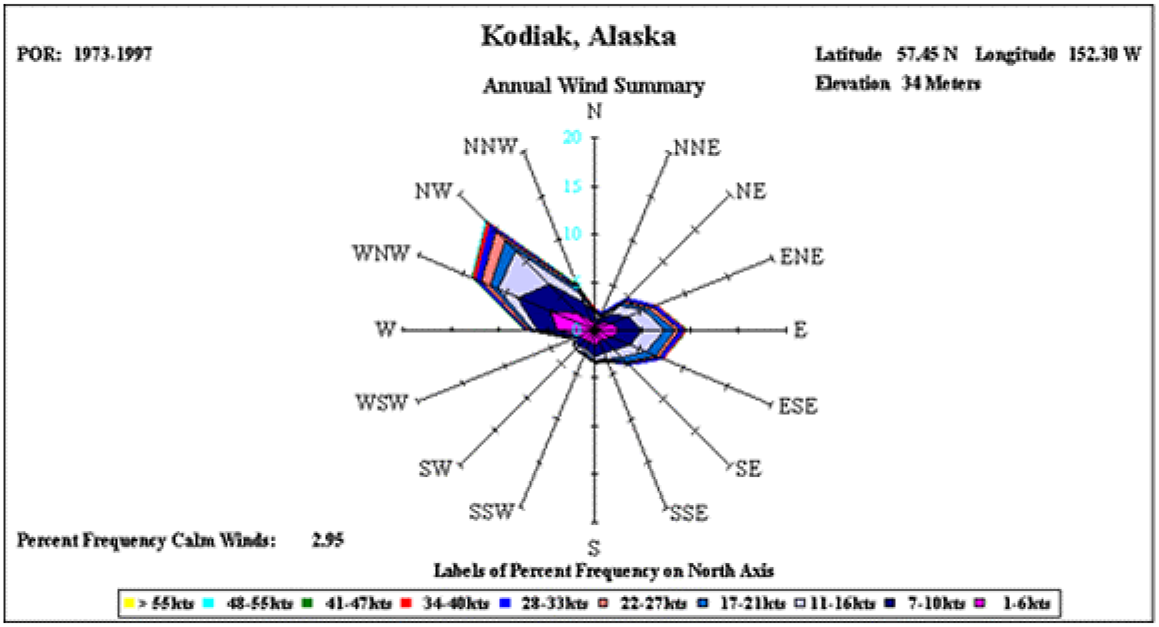

Figure 13. Wind Rose

This study obtained wind data recorded by a National Weather Service anemometer at the Kodiak Airport. The Air Force Combat Climatology Center (AFCCC) provided the data for a period from 1949 to 1996 for peak gust winds and 1973 to 2001 for 2-minute average sustained winds. AFCCC provided an extreme value analysis that gave wind speeds for various return periods and directions. Results of this analysis are shown in table 1.

Table 1. Extreme Wind Analysis Results for Kodiak, Alaska

2-minute average winds (mph)

	Return Period (years)							
Wind Direction	1.1	1.25	2	5	10	20	50	100
Northeast 30°-60°	34.3	36.1	39.4	42.5	44.1	45.4	46.9	47.8
Southeast 120°-150°	32.0	34.1	38.0	41.9	43.9	45.5	47.3	48.5
East 75°-105°	36.0	37.6	40.9	44.4	46.4	48.1	50.0	51.3
North 345°-015°	27.2	29.2	33.3	37.6	40.1	42.1	44.5	46.1

Source: Air Force Combat Climatology Center, period of record (1973-2001)

Wind speeds at the city of Kodiak airport and at Port Lions may not necessarily correlate as being the same. They are likely similar in that higher speed winds would be generated by the same storm systems moving through the area. However, topographic effects could cause wind speeds at Port Lions to be higher that at the city of Kodiak, particularly during northeasterly storms. Northeasterlies would tend to come straight in off the open ocean at the city of Kodiak while at Port Lions, they would be channeled somewhat by mountains rising above the shoreline of Marmot Bay and propagate toward the harbor and the town. Such constriction of winds would tend to increase the wind speed. Local residents of Port Lions report that winds are generally more intense there compared with Kodiak city.

In summary, for purposes of this study, design winds selected for use in derivation of the design waves are taken from the hindcast done by WES in 1982 previously discussed. Such wind speeds are taken to be representative of the 50-year design wind for the northeast direction (30° to 60°). The design wind speed of 89 mph with a duration of 3.8 hours was used. Using methods described in the 1984 *Shore Protection Manual* (SPM), this design wind equates to a one-hour wind speed of 81.4 mph. For the southwest direction (215° to 245°) a one-hour wind speed of 50.0 mph was determined.

2.3 Tides and Currents

The tides at Port Lions are generally diurnal with two highs and two lows occurring daily. Tide levels, referenced to mean lower low water (MLLW), are shown in table 2. Extreme high water levels result from the combination of astronomic tides and rises in local water levels due to atmospheric and wave conditions.

Table 2. Tide Elevations, Port Lions, Alaska

Water Level	Elevation, ft, MLLW
Est. Highest Tide (observed)	+14.0
Highest Tide (predicted)	+12.3
Mean Higher High Water (MHHW)	+9.6
Mean High Water	+8.7
Mean Low Water	+1.1
Mean Lower Low Water (MLLW)	0
Lowest Tide (predicted)	-4.0

Source: NOAA National Ocean Service

The regional currents in Settler Cove are driven primarily by tides and partially by wind. Discharge from the local creeks in the area also affects currents in the bay during high flows. In general, maximum current velocities have been estimated to be about 2.5 knots on the ebb tide. Significantly lower current velocities are estimated for the flood tide. A circulatory pattern in a counterclockwise direction has been observed in Settler Cove. Local residents substantiate this pattern by reporting that when ice is present in the area, it disperses in the same direction. Surface drift is also indicated by accumulation debris above the high tide line along the southeastern shoreline and at the head of the bay.

The wind driven component of the currents in the project vicinity is variable and depends on wind velocity. Due to the shallow water depths at the head of the bay, wind driven currents may represent a significant factor in the overall current regime of Settler Bay.

Wave generated currents may also be a component in the bay's water circulation. During strong northeasterly winds, wave action may drive the counterclockwise current pattern as water is stacked up along the western shoreline. Littoral currents in a southerly direction at the existing breach in between the breakwaters are indicated by the buildup of sediments and driftwood. This pattern may continue south of the harbor as waves enter the bay around the head of the breakwater and propagate toward the town of Port Lions.

2.4 Storm Surge

Storm surges are increases in water surface elevation caused by a combination of relatively low atmospheric pressure and wind-driven transport of seawater over relatively shallow and large unobstructed waters. Storm-induced surges can produce short-term increases in water levels to an elevation considerably above mean water levels. Storm surge at Port Lions has not been studied in depth; however, indications are that the area does not experience significant storm surges. Rugged terrain onshore and steep bathymetry offshore are conditions that preclude high storm surges. Highest surges are likely to be on the order of 3 ft or less in addition to wave set-up and tides during extreme low-pressure events. Typically, storm surges at Port Lions would be expected to be less than 1 ft. As table 2 shows, tides at Port Lions are the major factor in the fluctuations in water surface elevations. The wind-driven transport of seawater is the second most important factor, followed by wave set-up.

2.5 Rivers and Creeks in Project Vicinity

Several small creeks drain off of the eastern slope of the mountains surrounding Port Lions into Settler Cove. These are relatively small contributors of sediments to the waters in the area due to low flows throughout most of the year. At the southern limit of the Cove, two main creeks converge with tidewater. Varying sediment loads are indicated by the alluvial fan that forms the back portion of the Cove. A broad shallow shelf is present south of the boardwalk. Clear water conditions generally indicate that these creeks are not of glacial origin. However, during high rainfall and breakup events, considerable sediment loads may be present. Much of this coarse material is deposited in shallow water immediately south of the boardwalk, relatively little accumulation of sediments along the shoreline farther north has occurred. At the existing harbor site, no creeks drain directly into the immediate area.

Three small creeks drain into Settler Cove north of the existing harbor. No indication of significant alluvial fan deposits is evident. It is presumed that sediments that are introduced to the shoreline at this point are carried south along the shoreline by littoral currents and are built up at the breach area between the breakwaters at the harbor.

2.6 Soil Conditions

General information about the soil conditions at the existing harbor site indicates relatively shallow bedrock offshore. Materials are generally poorly graded gravelly sands, underlain by weathered greywacke. The beach gravels are plately with individual particles about one half inch thick by two inches in diameter grading down to sand sizes. The field classification of beach materials is sandy gravel. There are sand and gravel deposits along the immediate shoreline at the site but they are somewhat limited. Exposed bedrock is evident at the site along the shoreline, particularly south of the existing breakwater.

The offshore materials at the site were characterized by a geotechnical investigation conducted by the Corps of Engineers in 1973. Boring logs and general description of conditions is presented in the 1976 Feasibility Study for Port Lions Harbor by the Alaska District Corps of Engineers. Subsurface materials appear to be mostly gravelly sand, weathered greywacke, and minor amounts of volcanic ash.

A complete description of soil conditions at the existing harbor site is contained in the Geotechnical Appendix of this report.

2.7 Sedimentation - Littoral Drift

The primary source of sedimentation at the existing harbor site is from littoral drift from the north where the shoreline is exposed to high wave energy at a very oblique angle. The sediment sources would include discharge from roughly two square miles of drainage collected in streams that empty onto the beach about ½ mile upstream of the harbor. There is also about two miles of shoreline erosion that would combine with the stream sediment to make up the composition of littoral drift from the north. It is assumed that the contribution to the sediment load from shorelines north of Talnik Point would be small.

Over the past 20 years the breach in the existing breakwater has shoaled in about 10 ft. Calculating the total volume change from the pre-construction survey to a more recent one can provide a good estimate of the total long-shore transport from the north. Most of the transport of coarse material will be along the upper shoreline and be deposited in the breach. There may also be finer sands moving in deeper water that could be deposited at the toe of the existing breakwater. If a sediment budget were to be analyzed for the harbor and adjacent shoreline it would be necessary to include the volume that may have been removed from the breach by the local community.

Based on the rate of shoaling within the breach, with some allowance for sediment removal and fine material in deep water, the long-shore drift can be estimated to be roughly 500 to 600 cubic yards per year. This average rate should continue since there is no evidence that the streams, shoreline characteristics, or wave energy affecting the long-shore transport have been altered.

The long-shore transport from within Settlers Cove needs to be considered also. This contribution to shoaling in the harbor will be small due to the protected wave climate and general lack of sediment sources. The dive that was coordinated by the Fish and Wildlife Service identified sand waves in the channel off the end of the breakwater. This is probably resident sand that is transported back and forth by combination of tidal currents and longer period waves. The total volume probably remains constant based on the equilibrium cross-section of the channel.

2.8 Ice Conditions

2.8.1 Settler Cove

Sea ice (pack ice) is absent in Settler Cove during the summer and winter months. In general, the waters in the vicinity of Kodiak Island are ice-free year round. Some local icing conditions along the shoreline and in the existing harbor can occur during extreme cold temperature periods. Strong low-pressure systems associated with storms in winter generally bring warmer temperatures that prevent the formation of significant quantities of ice for long periods of time. Some ice has been reported in the existing harbor area but it is relatively short lived. Photographs of the existing harbor area are available that do show significant ice accumulation in and around the float system. Open water, however, appears to be present in the entrance channel and offshore of the breakwater during such conditions.

Periods of high rainfall followed by rapidly falling temperatures can contribute to significant ice formation as well. Ice can form in protected bodies of water, such as harbors, if freshwater enters the harbor and wind, wave, and tidal action do not disperse it. At Port Lions, wave and current conditions as well as the short duration of cold temperature around the existing breakwater, ice does not generally persist for prolonged periods of time. It is recommended that any fresh water inflows directly into the harbor itself be minimized or eliminated to prevent or reduce ice formation and accumulation. On rare occasions, ice can form and remain for several weeks if cold temperatures are persistent such as that which occurred in 1999. Since seawater can also freeze at temperatures of 28 degrees F and lower, ice can form in the harbor even with no influence from fresh water sources.

2.8.2 Marmot Bay

Marmot Bay typically does not experience icing during the winter months under extreme cold conditions in the Kodiak Island area. Extreme cold temperatures are generally short lived in duration. Warmer temperatures associated with low-pressure storm systems and strong winds keep ice formation at a minimum.

3.0 WAVE ANALYSIS

3.1 Wave Climate

The wave climate in the Port Lions area can be characterized as being oriented in one of two directions depending on wind direction; either from the northeast, or from the southwest. Some open ocean swell (long period waves) can reach the existing harbor area, however the northeasterly waves that adversely impact the harbor are reportedly to be shorter period, locally generated waves. Open ocean swell that does reach the harbor has been observed to be a very low in amplitude and creates a slow horizontal motion at the floats. It is generally not problematic. The northern half of the Settler Cove shoreline is directly exposed to the northeasterly fetch across Marmot Bay and experiences moderately high waves under storm conditions. Such waves are generally in the 5- to 7-ft high range with periods of 4 to 5 s based on observations at the existing harbor site. During northeasterly winds, the existing harbor and surrounding shoreline is exposed to these waves propagating directly in from Marmot Bay. These waves do cause severe problems in the existing harbor. Part of the float system was destroyed in 1999 and numerous vessels have been damaged while tied up in their stalls.

Waves may also be generated from the southwesterly fetch toward town and propagate across the shallow water in the back of Settler Cove. These waves generally have not caused significant problems in the harbor, as they are very short period, low amplitude waves. Long-time Port Lions residents estimate the highest southwest highest waves in the 2- to 2.5-ft range with periods of 2 to 3 s.

The harbormaster at Port Lions has described the highest wave conditions impacting the seaward face of the breakwater at the harbor as 5 to 6-ft locally generated waves from the northeast. Waves come in toward the breakwater on a line of sight to the small islands out in Marmot Bay known as the Triplets. These waves enter the mooring area directly off the seaward end of the breakwater and impact the float system. They also appear to stack up water at the breakwater breach adjacent to the shoreline and surge into the harbor. Video taped storm conditions during November of 1999 appear to show this wave surge into the harbor at the breakwater breach.

3.2 Fetches

The shoreline of Settler Cove at the existing harbor is oriented generally in a northeast-southwest direction. The longest local fetch for the existing harbor is in the northeasterly direction at an azimuth of approximately 55°. Two methods for calculating this fetch were used in the 1982 Letter Report; the JONSWAP and SMB methods. The effective fetch was developed using the SMB method for use with the corresponding design wind. An average of nine radials at 3° increments was used. This analysis resulted in an effective fetch of 9.2 miles. The straight line fetch was developed using the JONSWAP method for use with its corresponding design wind. An average of three radials over a 10° arc was used. The analysis resulted in a straight-line fetch of 29.7 miles. A figure showing these two methods is presented in the Corps' 1982 Letter Report 1 for Port Lions Harbor.

The original fetch analysis was performed by the Corps of Engineer and presented in the 1977 Detailed Project Report. It resulted in an effective fetch of 16 miles for the northeast direction at an azimuth of 55°.

For this study, an effective fetch for the northeast direction was calculated using methods outlined in the 1984 Shoe Protection Manual (SPM). Nine radials at 3° increments were arithmetically averaged to determine this fetch length. Sensitivity to orientation of the central radial was explored and it was found that its position was quite important. Various orientations were tried by making fine adjustments to maximize the potential effective fetch. By trial and error, a central radial at an azimuth of approximately 57° appeared to result in the maximum fetch length to the northeast. The fetch length calculated was 18.8 miles. The nine radials were then rotated slightly to the north and their origin was shifted to the northeast of the project site. The fetch was recalculated to be 26.5 miles. While this fetch does not represent the direct line of site area for wave generation, it may be a more realistic representation of the actual over water area for wave generation that impacts the project site.

Marmot Bay is generally oriented in a northeast-southwest direction. It begins to open up toward the open ocean in an east-west direction with distance from the existing harbor at Port Lions. The available fetch for wave generation then becomes virtually unlimited with the Gulf of Alaska. However, northeasterly winds would tend to not generate additional wave action at the harbor from this portion of the open ocean. Rather, such wave energy would tend to impact the city of Kodiak's northern exposed beaches.

Longer period swell from the open ocean coming from the east direction would propagate toward Marmot Bay. Some refraction of this swell to the southwest toward the harbor is possible. The Harbormaster and several local harbor users indicate that such swell does not cause any significant problems in the harbor. A slow rolling motion has been observed in the harbor but it is very low in amplitude and does not cause significant damage according to local vessel owners.

For the southwest direction an effective fetch of 0.68 mile was calculated using SPM methods. Much of the back portion of Settler Cove goes dry on low tides, however a high tide condition was used for this fetch calculation. This would represent the worst-case scenario for winds from the southwest and is considered conservative. Table 3 provides a summary of the fetch distances determined in this study for the project site. Figures 14 and 15 show the layout of the fetch radials in the various directions of the wind for this study.

Table 3. Fetch for Port Lions Harbor

Direction	Fetch Distance (miles)
Southwest (220°)	0.68
Northeast (57°)	18.8
Northeast (47°)#	26.5

Fetches calculated per the 1984 SPM (9 radials at 3° increments). Northeast (57°) reflects local fetch and does not include effects from open ocean east to the Gulf of Alaska. Northeast (47°) calculated with origin just to the northeast of the project site.

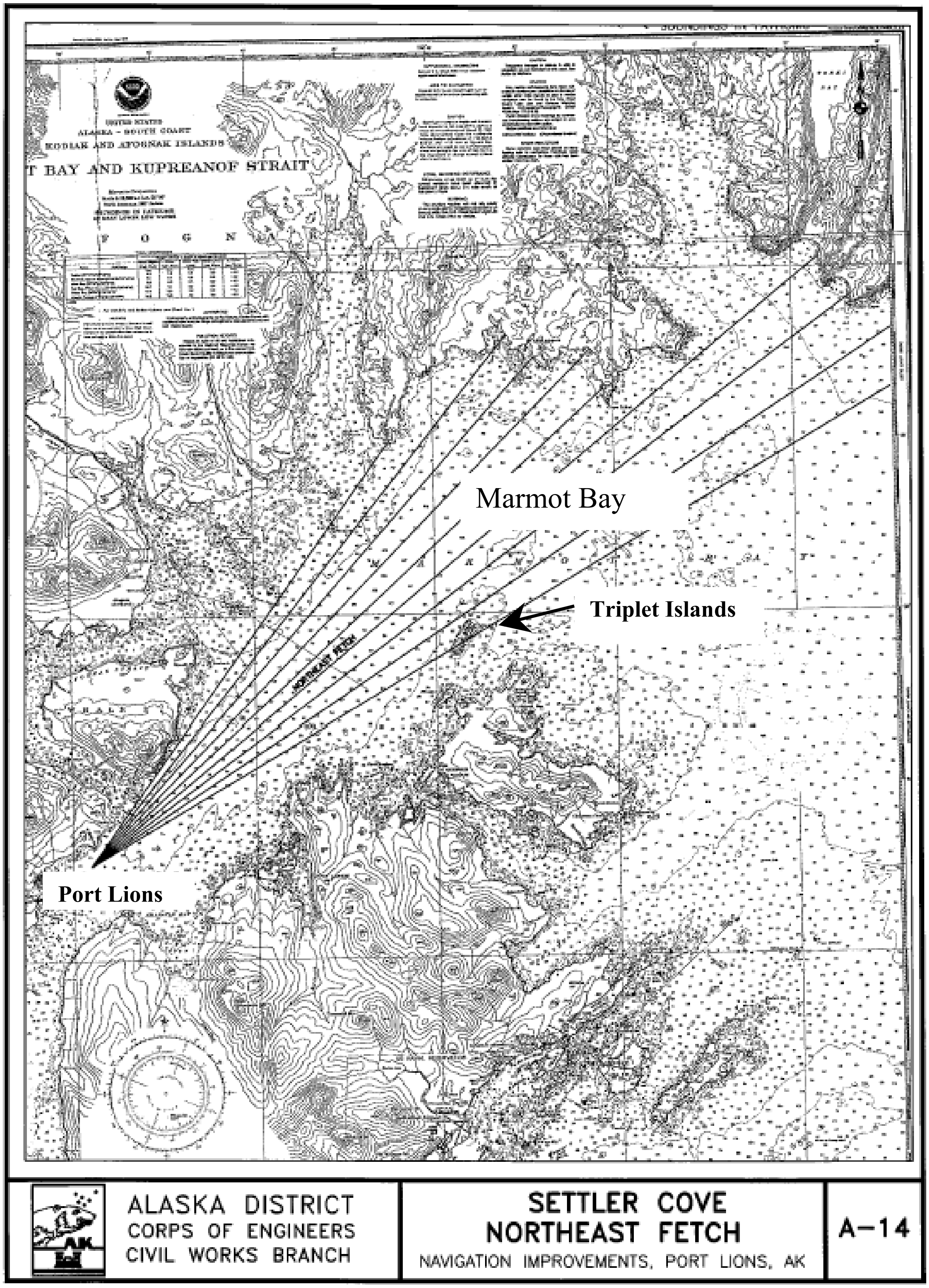

Figure 14. Settler Cove Fetches

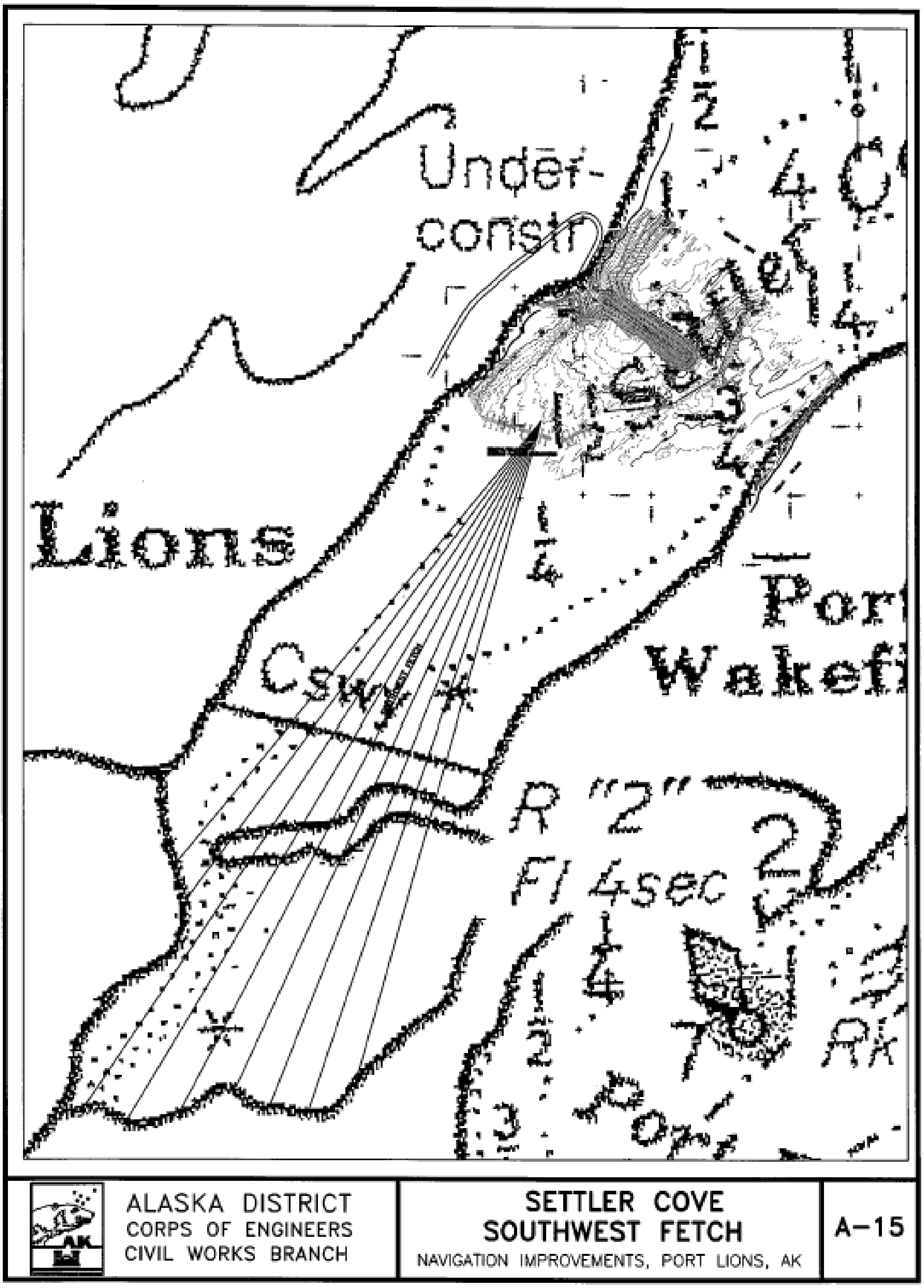

Figure 15. Settler Cove Fetches

3.3 Wave Prediction

The original wave analysis performed by the Corps of Engineers is presented in the 1977 Detailed Project Report for Port Lions Harbor. Using a design wind of 40 mph, an effective fetch of 16 miles, and correcting for refraction and shoaling, a design wave height of 4.5 ft with a period of 5 s was determined. This wave was interpreted to represent the 50-year design condition and was used for breakwater design and the diffraction analysis in the harbor basin.

Following partial failure of the breakwater after its initial construction, a subsequent wave analysis was prepared by the Alaska District and by WES for the Corps' Letter Report of June 1982. Using both the SMB and JONSWAP methods described earlier, estimates of the 50-year design wave were determined. These resulted in design waves of data shown below.

	Alaska District Frequency Analysis		WES Hindcast Analysis	
	JONSWAP	SMB	JONSWAP	SMB
Wave Height (ft)	10.8	7.5	14.5	8.0
Period (s)	8.0	6.0	8.5	6.0

The WES hindcast design wave height of 14.5 ft and period of 8.5 s was selected for further application to be conservative. A wave ray tracing computer program was then used to develop refraction and shoaling coefficients. This program (WAVE, 720[x[6[RICFO) calculates the above coefficients and plots wave orthogonals based on the WES design wave height and period. This program was developed by WES in the 1970's. Guidance for its interpretation was based on experience with the program along with Technical Report (TR)-4 and the 1974 SPM. In general, wave ray tracings would be expected to follow smooth contours in response to relatively smooth bathymetry. Where wave ray tracings cross or converge, a caustic condition is indicated. Depending on the site-specific conditions, it was generally interpreted based on experience either as an outlier or to represent an increase the wave height up to a maximum factor of 1.4. In general, where wave orthogonals converge, wave heights increase, where they diverge, wave heights decrease.

For Port Lions, the refraction and shoaling coefficients ($K_{r\,=}$ 0.605 and K_S = 0.941 respectively) were applied by WES and a final design wave height of 8.25 ft was determined. A rounded value of 8.0 ft for the design wave height was selected. The potential for breaking waves was investigated based on SPM methods. It was determined that non-breaking wave conditions would apply for depths and bottom slopes in the area of breakwater construction.

Predicted wave heights for this study were calculated using the 50-year design wind speeds presented in Section 2 of this appendix. Methods described in the 1984 SPM and the STWAVE numerical model were used to predict wave heights. Refraction and shoaling coefficients were applied as well. The design waves for the various directions were determined from the results of the two different prediction methods. The STWAVE results were used to take into account refraction, diffraction, and shoaling internally based on bathymetry and the complex shoreline geometry for comparison with the results from the SPM method.

The wind data used to model wave growth was transformed to more accurately reflect the boundary layer above the water surface. Wind speed depends on elevation, roughness of the surface over which the wind is blowing, and temperature gradients.

The SPM predicts wave heights based on fetch distances and wind speeds. The fetch distance and wind speed are used to determine whether the wave condition is limited by the fetch length or by the duration of the wind. STWAVE is a spectral wave energy propagation model that includes refraction, diffraction, and shoaling, but does not include reflection.

Shoreline and bathymetric conditions were defined by water depths and the locations of land into the STWAVE model at a specified grid spacing. Depths were obtained from the Geophysical Data System for Hydrographic Survey Data (GEODAS) database, which is essentially NOAA chart data, showing the bathymetry of the area. A grid was established for the northeast wave direction into Marmot Bay and out into the open ocean of the Gulf of Alaska. The grid origin (i=0, j=0) was located just west of Cape Izhut on Afognak Island with the positive x-direction to the west and positive y-direction to the south. Grid spacing of 100 meters by 100 meters was used. The model was run for the existing harbor site using the 50-year wind speed of 81.4 mph for the NE direction. The first test condition (1) used swell from the open Gulf in conjunction with local winds to estimate wave heights and periods at the project site. An incident wave of height of 26 ft and period of 16 s (similar to offshore design wave used for Kodiak Harbor project) was input at the ocean grid boundary at 0 degrees to the STWAVE coordinate system. A wind speed of 81.4 mph at an angle of 30 degrees to the coordinate system was used as input for the locally generated component of wave growth. The tide elevation was set at 0.0 ft MLLW. The second test condition (2) used the same parameters as (1) except the tide elevation was set at 9.6 ft MLLW, which represent mean higher high water (MHHW). The third test condition (3) used the same open ocean wave above with a zero wind speed to test the influence of swell alone at the project site. The tide elevation was set at 0.0 ft MLLW. The fourth test condition (4) used the same parameters as (3) expect the tide elevation was set at 9.6 ft MLLW.

The STWAVE model results showed that waves do propagate into the proposed site under the worst conditions with winds from the northeast and east. However, these waves are locally generated. The influence of swell from the Gulf of Alaska was shown to be minimal (approximately 1 ft in height and less) under the various test conditions. Wave heights and periods predicted by the model were compared with local observations for the existing harbor under extreme storm conditions. The model predicted maximum significant wave heights of approximately 5.7 ft with periods of 7.0 s at the grid points located just outside and adjacent to the existing breakwater. Such wave heights are slightly less, but do correlate fairly closely with local observations during the highest winds from the northeast and east. Indications are that the open ocean fetch to the east of the project area does not significantly contribute to the overall wave climate. The STWAVE analysis also did not give any indication of wave convergence or magnification just outside the harbor entrance as was suggested by the wave ray computer program performed by WES in 1982. This applies for the grid spacing specified.

Locally generated waves from the northeast and southwest represent the design conditions for the project. After detailed and evaluation of the various methods and analyses discussed above, the design wave conditions for this project were determined. Specifically, the interpretation of the WES wave ray tracing program output in conjunction with local observations and analytical methods lead to a high level of confidence in selecting the values to use for design purposes. A design wave height of 8 ft with a period of 8.5 s from the northeast direction was selected. A design wave of 2.5 ft with a period of 2.4 s from the southwest direction was selected.

Results of the wave analysis are shown in table 4. The wave heights calculated represent the significant wave height, H_s, which is the average of the highest one-third of all waves generated. The design waves shown in table 4 are for a design still- water level (SWL) of 9.6 ft MLLW. The design waves selected appear to correlate well with what longtime local residents have observed during extreme storm events in the Port Lions area.

The near-shore breach between the existing main breakwater and the existing stub breakwater allows unacceptably high wave energy into the mooring area of the harbor. An analysis of the incident wave height of 8 ft and period of 8.5 s showed that waves of approximately 2 ft in height enter the mooring area during large storms. Local harbor users as well as video of northeast storm events indicate that wave action causes a surge to occur through the breach. Actual wave heights are not so much of a problem inside the mooring area as the surge of water coming through the breach as it impacts the float system. An estimate of the magnitude of this phenomenon was performed using wave diffraction methods and by analyzing video of storm events. For design purposes, a 50-year wave height of 2 ft was selected.

A design wave height of 8.0 ft and period of 8.5 s was used for the breakwater reconstruction completed in 1983. Armor stone weight ranging from 4,800 to 2,900 lbs with a layer thickness of 6.5 ft was determined and used for the reconstruction. The breakwater has performed well and there has been no stone movement or loss since its construction over 20 years ago. Indications are that the design wave selected was appropriate for this site. Physical modeling would be recommended for justification of any proposed reduction in design wave height. Additional numerical wave, current, and water level modeling, deployment of wave buoys to collect wave height and period data in various locations, and collection of additional wind data would also be recommended to fully characterize the wave climate at the site if increases or decreases in design wave heights were proposed.

Table 4. Wave Analysis Results

	JONSWAP/SPM		STWAVE		Design Wave	
Direction	H_s (ft)	T (s)	H_s (ft)	T (s)	H_s (ft)	T (s)
Existing Harbor Site						
Northeast (55°)	8.0	8.5	5.7	7.0	8.0	8.5
Southwest (230°)	2.5	2.4			2.5	2.4

4.0 EXISTING HARBOR FACILITIES

4.1 General Description and Background

The existing harbor at Settler Cove is located north of the city of Port Lions on the western shoreline of the Cove. It currently has adequate space for vessels in the existing moorage area and can accommodate vessels up to 60 ft in stalls and up to 130 ft as transient moorage. The float system has been damaged to the point of being unsafe at the outer margin of the harbor. The southern-most float was destroyed and removed during the storm of 1999. The mooring area depth average -15 ft MLLW and is adequate for the current fleet. An area of approximately 5.3 acres (ac) is available for mooring. Sufficient maneuvering and turning areas are also present in the existing harbor.

A 150-ft wide dredged entrance channel to a depth of –14 ft MLLW accommodates access to the harbor around the eastern tip of the existing main breakwater. The length of the entrance channel is 1,030 lineal ft. The eastern limit of the harbor is fully open to wave action from the northeast. This exposure has caused damage to vessels and the float system and has also created hazardous navigation conditions inside the harbor during storms.

A dock, boat grid, and launch ramp are located inside the harbor to support the fishing fleet and transient vessel traffic. The small beach area in the breach between the breakwaters is available for use as a loading/unloading location for barges.

An existing dock is located in Port Wakefield to the east of the city of Port Lions. This facility has been used in the past for fish processing operations. Its current use is now limited to cargo unloading and unloading as well as the terminal for the State Ferry.

4.2 Existing Harbor, Settler Cove

The existing harbor facilities at Port Lions are shown in Figure 1 of the main report. The Corps of Engineers constructed the original breakwater in Settler Cove in 1981. The project consisted of a single detached 600-ft long rubblemound breakwater, an attached 170-ft long rubblemound stub breakwater, and an armored staging area adjacent to the harbor. A breach was left between the breakwaters to facilitate tidal circulation in the harbor.

In November of 1981, a severe storm in the Gulf of Alaska generated high winds and waves from the northeast direction. Much of the seaward face of the newly constructed main breakwater was damaged. It was estimated that approximately 4,000 cubic yards of armor stone was displaced from the breakwater and deposited in the toe area.

The main breakwater was reconstructed in 1983 with larger armor stone and a flatter (2H:1V) seaward side slope. The breakwater was also lengthened by 125 ft from the original head location. This required dredging a 150 ft wide entrance channel around the new head location to a project depth of –14 ft MLLW. The mooring facilities were then constructed in subsequent years by the State of Alaska. Pilings for the float system were drilled and socketed due to shallow rock foundation conditions within the harbor. Since then, storm induced waves from the northeast direction have been problematic to the outer portions of the float system and vessel moored in the harbor. Northeast waves diffract around the head of the main breakwater and directly impact the floats.

Three segments of floating breakwater were installed along the southern limit of the harbor in 1999. The intent was to shield the harbor from damage causing wave action as much as possible. These floating breakwaters are reinforced concrete, post-tensioned, ladder type design structures and are owned and maintained by the ADOT&PF. They were surplus from Kodiak Harbor after the Corps of Engineers completed a project there in the mid-1990's. According to local harbor users, these floating breakwaters have provided good protection from southwesterly waves. However, they appear to have made conditions worse in the harbor when waves are coming in the entrance channel from the northeast direction. Video taped wave action from the November 1999 storm event indicate that northeasterly waves may have been redirected into the mooring area and focused on the float system. Local observers have reported that this may be occurring under northeast storm conditions. The ADOT&PF subsequently attempted to decrease the wave conditions in the harbor by re-positioning the floating breakwaters. Adverse conditions have remained and portions of the float system have been destroyed as a result a numerous storm events.

4.3 Port Wakefield Dock

The facilities at Port Wakefield consist of a 400-ft long timber dock with a 40-ft wide working deck at its terminus. Remnants of the old fish processing facilities are also present. Cargo for the community is unloaded at this facility by freight vessels and barges. The State Ferry also calls on the community at this location.

The site is connected by road to the city of Port Lions less than 5 miles away. It also has pedestrian access via the boardwalk that reduces the distance to town to about 1 mile. A launch ramp and small parking area are also at the site.

Water depths are typically in the 20- to 30-ft range. Such depths can accommodate the larger freight vessels that use the facility. Facilities for mooring vessels are limited and no permanent slips are available. Vessels can anchor up offshore in the lee of the dock.

5.0 HARBOR DESIGN CRITERIA

5.1 Design Vessel and Fleet

The economic analysis for this study analyzed the vessel demand for the existing project. The fleet considered for the various alternatives is described in Appendix B, Economic Analysis. Lengths, beams, and drafts for these vessels were developed in conjunction with the harbormaster. Proposed harbor plans were laid out to accommodate the identified fleet. The design vessel (the largest vessel in the fleet) for the harbor is 58 ft long with a beam of 19 ft and a draft of 6 ft. Vessels such as barges and larger transients occasionally call on the harbor.

5.2 Wave Height Criteria for the Entrance Channel

Breakwaters for the proposed alternatives were positioned to reduce wave heights in the harbor entrance and mooring area. Due to the orientation of the entrance channel into the predominant wave direction, wave energy would still propagate into the entrance channel itself. Reduction of wave heights to a maximum height of 2.5 ft at the inside limit of the entrance channel should be achieved with the proposed layouts. Progressively smaller wave heights down to 1 ft and less were allowed into the mooring area. Such wave heights would not impact vessels entering and leaving the harbor and will eliminate damage due to wave action in the mooring area.

5.3 Wave Height Criteria for the Mooring Area

The ADOT&PF provided the wave height criteria for the mooring area. The criteria shown in table 5 summarize the wave heights and horizontal motion considered for the mooring basin design. Such criteria closely follow “Planning and Design Guidelines for Small Craft Harbors” (ASCE, 1994). A maximum allowable wave height of 1 ft in the mooring area was used for a 50-year incident design wave event. This criterion is generally used by the Corps of Engineers and parallels that outlined in EM 1110-2-1615, "Hydraulic Design of Small Boat Harbors," which represents many years of experience in harbor design. This criterion is appropriate to capture the economic benefits for the fleet by adequately minimizing damages.

Since long-period ocean swell (i.e. wave periods greater than 6 s) reaches the project site but does not cause significant problems in the harbor, criteria for horizontal motion were not applied. Minor horizontal motion is expected in the proposed protected harbor, however it will be somewhat reduced compared to the current conditions. Complete elimination of long period swell induced horizontal motion may not be achievable regardless of design layout of the harbor.

Diffraction diagrams from the SPM were used to calculate wave heights expected in the harbor alternatives. Wave heights were determined by multiplying the incident design wave height by the diffraction coefficient K’ from diffraction diagrams. The 1-ft wave height criterion was used for the alternatives considered.

Table 5. Wave Criteria for Mooring Basin

Recurrence, Orientation, and Period	Good	Excellent	Moderate
For wave heights (H1/3):	(ft)	(ft)	(ft)
1 year interval, Beam Sea,T>6	0.5	0.4	0.6
1 year interval, Beam Sea,2<T<6	0.5	0.4	0.6
1 year interval, Beam Sea,T<2	1.0	0.75	1.25
50 year interval, Beam Sea,T>6	0.75	0.5	1.0
50 year interval, Beam Sea,2<T<6	0.75	0.5	1.0
50 year interval, Beam Sea,T<2	1.0	0.75	1.25
1 year interval, Head Sea,T>6	1.0	0.75	1.25
1 year interval, Head Sea,2<T<6	1.0	0.75	1.25
1 year interval, Head Sea,T<2	1.0	0.75	1.25
50 year interval, Head Sea,T>6	2.0	1.5	2.5
50 year interval, Head Sea,2<T<6	2.0	1.5	2.5
50 year interval, Head Sea,T<2	2.0	1.5	2.5
For horizontal motion (ft):			
1 year interval, Beam Sea,T>6	1.0	0.75	1.25
50 year interval, Beam Sea,T>6	2.0	1.5	2.5
1 year interval, Head Sea,T>6	2.0	1.5	2.5
50 year interval, Head Sea,T>6	4.0	3.0	5.0

5.4 Entrance Channel, Maneuvering Area, and Mooring Basin Design

The required entrance channel width was determined using criteria given in EM 1110-2-1615 “Hydraulic Design of Small Boat Harbors” (USACE 1984), in "Planning and Design Guidelines for Small Craft Harbors" (ASCE 1994), and in the State of California's "Layout and Design Guidelines for Small Craft Berthing Facilities" (1980). For a two-way channel with 2.5- knot currents, the recommended width should be 180 percent of the beam of the design vessel, plus an additional 80 percent for traffic clearance and 60 percent for breakwater clearance. An additional allowance of 20 percent for adverse wind, wave, and current conditions was included given the severity of storms at Port Lions. Also, due to the occasional use of the harbor by larger vessels and barges, and additional allowance of 15 percent is included. Therefore, for the proposed entrance channel, a total bottom width of 100 ft was calculated and would allow adequate maneuverability and clearance on each side of the breakwaters.

The maneuvering and mooring area widths were designed during the original harbor feasibility study so that there would be adequate room for vessels to turn and dock. With the actual float system layout that was constructed, widths for turning and docking are more than adequate for the fleet that uses the harbor. The existing float system is laid out such that vessels have no problem entering the float stalls to tie up.

Evaluation of Float System - The existing float system was evaluated by the ADOT&PF. In 1983, the Corps completed the single breakwater at Port Lions; the State completed the inner harbor facilities in about 1986. The State has removed most of C float, about 1/3 of B float and continuously repairs hinges and thru rods in the facility to keep it in fair condition. The sections removed from the water are stored near the harbor office. The float system is less than 70% useable. High maintenance costs and continued storm damage have left the harbor nearly empty for much of the year.

Overall, the condition of the existing float systems can be characterized as poor to failed. Once the basin protection is improved, the sponsor will rebuild the inner harbor floats.

5.5 Depths

The required entrance channel depth was calculated as follows:

Entrance channel	
Vessel draft	-6.0 ft
Pitch, roll, and heave, based on 1/2 of the wave height in the channel	-1.5 ft
Squat	-0.5 ft
Access (tide)	-4.0 ft MLLW
Safety clearance (based on sand/gravel bottom)	-2.0 ft
Total	-14.0 ft MLLW

The existing entrance channel was originally designed to have a project depth of –14 ft MLLW. The June 2002 condition survey indicated that the existing entrance channel depths vary from –14 ft to –16 ft MLLW. The required depth for the maneuvering and mooring area within the existing harbor was calculated as follows:

Maneuvering/mooring area	
Vessel draft	-6.0 ft
Pitch, roll, and heave	-0.5 ft
Squat	-0.5 ft
Access (tide)	-4.0 ft MLLW
Safety clearance (based on sand/gravel bottom)	-2.0 ft
Total	-13.0 ft MLLW

The existing depth in the maneuvering/mooring area varies from –13 ft to –18 ft MLLW in the main part of the harbor. Depths for the smaller draft vessel such as skiffs which use the small stalls along the western edged of the harbor vary from –10 ft to –12 ft MLLW. Such depths are adequate for the fleet.

Since the existing natural depths are sufficient for the entrance channel and maneuvering/mooring areas throughout the range of tide elevations, use of the lowest tide in determining the proposed depths are economically justified. There are no costs associated with the variation of a range of tidal depth criteria.

The natural depths offshore of the existing harbor vary from –20 ft MLLW at the seaward limit of the entrance channel down to –50 ft MLLW and greater with distance to the northeast into Marmot Bay. Commercial fishing vessels may enter the existing harbor basin loaded at all tide stages. Loaded drafts were used to calculate required depths for the entrance channel, maneuvering area, and mooring area.

5.6 Entrance Channel Depth Optimization

Optimization of the entrance channel depth was performed during the 1977 Feasibility Study for Port Lions Harbor. Based on the drafts of vessels in the fleet, a range of channel depths and the percentage of time the channel would be accessible were analyzed. An entrance channel depth of –15 ft MLLW was determined to be the optimum channel depth based on percentage of time accessible, costs for construction, and economic benefits. The project was then authorized with such an entrance channel depth. The entrance channel depth was then reanalyzed for the 1982 Letter Report 1 for the Port Lions breakwater repair project. Since extension of the existing breakwater would require dredging an entrance channel around the head, the required depth was held at –15 ft MLLW.

No change in the existing entrance channel depth is proposed in this study for the proposed project. Existing depths are sufficient for the fleet that currently uses the harbor. None of the alternatives considered require dredging. The existing entrance channel would depth would remain the same.

5.7 Floating Breakwater and Wave Barrier Design Considerations

Floating breakwaters reduce wave action by reflecting the incident wave and by dissipating some of the wave energy through friction and turbulence. Wave barriers reduce waves more by reflection than by turbulence. Some of the incident wave energy passes through both floating breakwaters and wave barriers resulting in a transmitted wave. The height of the transmitted wave is calculated as follows:

$$H_t = C_t * H_i$$

where H_t = transmitted wave height
C_t = transmission coefficient
H_i = incident wave height

The transmission coefficient is greatly affected by the width of the floating breakwater compared with the wavelength of the incident wave, and the draft of the breakwater compared with the depth of water. Transmission coefficients for wave barriers are a function of the depth of the barrier, the depth of water, and the wavelength of the incident wave.

The transmitted wave is also affected by the angle at which the incident wave impacts the breakwater. The waves inside the harbor are a combination of the transmitted wave and the waves diffracted around the ends of the breakwater. This can be expressed by the following equation:

$$H = \sqrt{H_t^2 + H_d^2}$$

where H = the wave height inside the harbor
H_d = diffracted wave height

For this project, floating breakwater and wave barrier design concepts were considered. At the existing harbor site in Settler Cove, design wave heights and periods for the northeast direction exceed the criteria for economically viable floating breakwater applications. Costs associated with very wide and deep draft floating structures preclude use of such designs. The wave barrier design concept also has limitations in economically reducing wave energy to acceptable levels. High costs for construction due to shallow bedrock sub-bottom conditions is the main factor that renders the wave barrier design inappropriate for this site.

Floating breakwaters are, however, appropriate for use as wave protection from the southwest direction. Wave heights and periods are within the range where such designs are applicable. Water depths of up to 15 ft MLLW allow use of floating breakwaters with bottom anchors or piles for positioning. Bottom anchors would very likely be more cost effective than piles due to shallow bedrock.

5.8 Water Quality and Circulation

Water quality and circulation criteria were applied to the alternative designs to minimize environmental degradation associated with harbor improvements. USACE 1993, EM 1110-2-1206 Environmental Engineering for Small Boat Basins is the established engineering guidance that forms the basis for addressing water quality and circulation in small boat harbor design. Nece, et al, 1979 "Effects of Planform Geometry on Tidal Flushing and Mixing in Marinas" is adopted as standard practice for estimating harbor basin flushing by use of an average exchange coefficient for one tidal cycle. This work is based on physical model studies of harbor basins of varying geometry and a tidal range typical of Puget Sound in the State of Washington. It is noted that the mean tidal range for the project site at Port Lions (9.6 ft) is greater than that for the Puget Sound area (6 ft). Flushing coefficients can be approximated by the tidal prism ratio: the difference in basin volume at high tide and low tide divided by the basin volume at high tide. It has been determined that average spatial values greater than 0.30 will provide for acceptable harbor basin flushing. It is also recommended that no more than 5 percent of the basin have values less than 0.15. The areas of possible low tidal prism ratios would be in the corners of the basin and should therefore be checked to ensure they meet this minimum value. Gently rounding the corners of the basin is also recommended to achieve the most efficient use of water area and promote water circulation.

Another criterion for water quality and circulation is the aspect ratio of the basin. This value is a measure of the length divided by the width of the basin. Generally, aspect ratios of greater than 0.3 and less than 3.0 are desirable. Such geometry will minimize possible zones of stagnation and short-circuiting of circulation cells within the basin. Additionally, the ratio of the basin planform area (A) to the entrance cross-sectional area (a) is recommended to be less than 400 for an optimal basin configuration for flushing.

All alternatives considered in detail for this study described in Section 6 used the above criteria for design and evaluation.

For proposed harbor improvements with floating breakwaters, the above criteria do not directly apply since the mooring area would not be fully enclosed as in a conventional harbor configuration. Floating breakwaters have been generally accepted as less detrimental to circulation and water quality however there are no criteria or research available as guidance. Stagnation zones that would cause deterioration of water quality would not be anticipated.

5.9 Uplands

The ADOT&PF requires that harbors in Alaska have a minimum uplands to total harbor area ratio of 0.40. This criterion was used as a general guideline for the proposed harbor improvements. Upland uses would include vehicle parking, boat and trailer storage, harbormaster's office, restrooms, and harbor support facilities.

There are approximately 1.5 ac of existing uplands available adjacent to the harbor for such usage. Since existing uplands are adequate for current needs and future projections based on the design fleet, creation of additional uplands would not be necessary for the proposed project. The 0.40 ratio of uplands to total area requirement is therefore waived for alternatives considered in this study.

The ADOT&PF also requires that non-point source pollution control measures be addressed in harbor design. The following summarizes their analysis and recommendations for this study:

Alternative designs for improvement of the Port Lions facility considered all aspects of non-point source pollution. Upon completion, the community of Port Lions will maintain and operate the small boat harbor. They will be encouraged to follow best management practices for harbor operation.

There is a small upland area adjacent to Port Lions Harbor. The area will remain as a gravel lot. There is no upland hull-maintenance area; none is likely. No change to the existing upland area is probable.

6.0 ALTERNATIVES CONSIDERED IN DETAIL

6.1 General

A wide range of alternatives was considered for navigation improvements at Port Lions. A matrix of possible alternatives for consideration was developed in the initial phase of the study that included various configurations of rubblemound and floating breakwaters. This phase narrowed the alternatives to three basic concept alternatives: one with an offshore detached rubblemound breakwater to the northeast and a floating breakwater to the southwest, one with an offshore detached rubblemound breakwater to the northeast and a rubblemound breakwater to the southwest, and one with an inner detached rubblemound breakwater to the east and southwest. Several minor variations of these concept alternatives were analyzed and refined to define the six alternatives considered. No sites other than the existing harbor site were explored in detail for consideration.

The alternatives were evaluated using established design guidance given in the appropriate Corps of Engineers Engineering Manuals (EM's), the SPM, and the Coastal Engineering Manual (CEM). Physical modeling of the alternatives was not included in the scope of this analysis.

After a thorough evaluation of the wave climate in Settler Cove, it was determined that rubblemound breakwaters for protection from the northeasterly wave exposure and floating or rubblemound breakwaters for protection from the southwesterly wave exposure were most appropriate and cost-effective. Relatively shallow water depths lend themselves to economically constructed rubblemound breakwaters for the project.

Vessel traffic conditions, including existing dock and barge operations, were considered in the layout of proposed alternatives. Development of an expanded harbor at this site would not impact current operations at the dock and barge landing. Vessels would continue to be able to maneuver and moor at both the dock and floats within the existing harbor and coexist with the increased vessel usage in the area.

The site has limited uplands but they are sufficient to support the fleet and associated harbor operations. Creation of additional uplands would not be necessary. This site also represents the most practical site for harbor development due to its relative proximity to the town of Port Lions. Other sites farther out past the airstrip or across the peninsula to Port Wakefield have the disadvantage of being located more than 5 miles from town.

6.2 Existing Harbor Site

The proposed harbor expansion is immediately adjacent to the existing harbor northeast of the town of Port Lions and has natural bottom elevations that range from –10 to –16 ft MLLW. Such depths in the area of the proposed harbor are suitable for cost effective rubblemound breakwater construction. The wave climate for the two directions of exposure is also suitable for cost effective rubblemound breakwater construction. The southern limit of the site may be applicable for floating breakwater protection also. A rubblemound breakwater structure would be required for wave protection from the northeast direction and would make use of the relatively shallow depths offshore. Many different harbor configurations were considered and optimized to determine the most effective and least costly alternative at this site. Optimum locations for the breakwaters were determined so that the quantities of material were reasonable for the size of the basin to be protected. The alternative plans at this site for a 50-year design life were laid out using breakwater alignments to protect the proposed maneuvering area, and mooring basin. Alternatives were numbered based on the initial screening numbering system used early in the study.

6.2.1 Alternative 1A.

This alternative, shown in figure 16, incorporates the following: a 700-ft long detached rubblemound breakwater located northeast of the existing breakwater, 732 lineal ft of concrete floating breakwater, a 40-ft long extension of the existing breakwater to the west for reduction in the existing breach width, and a 75-ft long extension of the existing stub breakwater at the barge landing to further reduce the breach width. The existing mooring basin would remain unchanged with this alternative. The 8.5-ac mooring basin could accommodate the range of vessels in the fleet with stalls oriented with the prevailing wind direction as at present. The existing float system could be expanded considerably in the future if so desired and still be protected from the northeast wave exposure. The harbor entrance would be oriented with more of an “S-turn” movement around the heads of the new and existing breakwaters and into the maneuvering area. This entrance channel configuration is somewhat different from the existing condition but was designed to meet safe navigation criteria under extreme wave and tidal current conditions. A new navigation marker light would be established along with the existing one to guide mariners into the harbor. The new floating breakwaters would replace the existing ones. Their orientation would be slightly modified to provide full wave protection from the southwest direction.

Harbor Basin. The harbor basin would not require dredging since existing depths range from –10 to –18 ft MLLW. These depths are sufficient for the design fleet based on criteria given in Section 5 of this appendix. The deeper portion of the mooring basin would be located nearest the entrance channel. The shallower portion would be located farther into the harbor toward the western shoreline. The maneuvering area just inside the basin would not require dredging since existing depths range from –12 to –17 ft MLLW. A total combined maneuvering and mooring basin area of approximately 10.0 ac would be available in the basin for alternative 1A. This area could easily be expanded in the future without additional breakwater protection or dredging.

Wave Heights. This alternative would meet the wave criteria established in Section 5 of this appendix along the floats inside the harbor basin. The breakwaters were positioned to reduce incident wave heights from the various directions of exposure to acceptable levels. The maximum wave heights in the mooring areas, based on the 50-year design incident wave, were calculated to be 1 ft and less. Progressively smaller wave heights would occur farther into the harbor mooring area, as shown in the diffraction diagrams in figures 17 and 18. Predicted wave heights inside the harbor under design conditions are calculated by multiplying the incident design wave height by the coefficient (K’) indicated. All directions of wave exposure were taken into account in determining the highest wave heights in the mooring area, however, the northeast direction was the most critical.

Circulation. This alternative would not enclose a basin proper since the proposed rubblemound breakwater would be located outside and offset from the existing harbor. It is estimated that the exchange of water in the harbor mooring area would be similar to that of the existing harbor during each tide cycle. The aspect ratio of the basin is 1.2. The ratio of the basin planform area (A) to the entrance cross-sectional area (a) is 61. The areas of potentially low exchange were checked to ensure that no more than 5 percent of the total area had exchange coefficients less than 0.15. All parameters meet the harbor design criteria for water quality and circulation.

Shoaling. Shoaling of the entrance channel would not be expected since there is no evidence of significant shoaling of sediments at the existing entrance channel. There are no significant sources of sediment such as major rivers or creeks in the area. A small fillet of gravel and sandy material is present along the shoreline at the existing breach indicating some accumulation of material from the northeast direction. The eastern shoreline is rocky and fairly abrupt with little accumulation of sediments. This material would not be expected to reach the entrance channel or mooring basin. The existing entrance channel has not required maintenance dredging and is not expected to with this alternative.

Construction Dredging. No dredging would be required for Alternative 1A.

Maintenance Dredging. Maintenance dredging would be expected to be minimal or not necessary at all in the future. Dredging has not been required in the existing harbor since its initial construction and reconstruction after the main breakwater was damaged. Littoral transport of sediments generally appears to be from northeast to southwest along the shoreline and the existing breakwater. The source of much of this material is believed to be dredged material from the initial dredged entrance channel disposed of on the seaside of the main breakwater. A decrease in deposition of this material has been observed every year to the point of being minimal at present.

Breakwaters. The positioning of the new rubblemound breakwater would create an entrance channel alignment allowing access from the northeast to the basin. Maximum depths of water are –18 ft MLLW along the alignment of the breakwater at the head. Foundation materials would be sand, gravel, and rock that would serve as a suitable base for the rubblemound structure. The existing main breakwater would be extended to the west to reduce the width of the existing breach along the western shoreline. A breach of 30 ft would be retained at an elevation 5 ft MLLW from the toe of the breakwater extension to the toe of the shoreline riprap. This would provide for continued fish passage along the shoreline and through the harbor.

Concrete box type floating breakwater segments would be constructed and replace the existing concrete floating breakwater segments. They would be positioned at depths of approximately –11 ft MLLW along the southwestern boundary of the mooring basin.

Rubblemound Breakwater Design. Methods described in the SPM using Hudson's equation were used to determine armor stone sizes for the new rubblemound breakwater and existing breakwater extension. Stone size for the rubblemound breakwater was determined using the significant wave heights presented in table 4, along with a sea-side side slope of 2H:1V and harbor-side slope side slope of 1.5H:1V, and a K_d value of 4 for a non-breaking wave. A stone specific gravity of 2.72 was used in the calculations. Armor stone (A1 rock) with a range of sizes from 6,100 lb maximum weight, 4,900 lb average weight, to 3,650 lb minimum weight would be used on the face of the breakwater. Secondary stone (B1 rock) would range from 3,650 lb maximum weight, 490 lb average, to 360 lb minimum weight. Core1 material would range from 360 lb maximum, 49 lb average, to 1 lb minimum. Armor stone thickness would be 6.5 ft, and secondary stone thickness would be 3 ft. The armor stone on the existing main breakwater has an average weight of 3,850 lbs. Since the main breakwater has been stable since its reconstruction in the early 1980's, the existing stone size is appropriate for the breakwater extension to constrict the fish passage breach. This armor stone (A4 rock) would have a range of sizes from 4,810 lb maximum, 3,850 lb average, to 3,080 lb minimum weight. Secondary stone (B4 rock) would range from 3,080 lb maximum weight, 385 lb average, to 300 lb minimum weight. Core4 material would range from 300 lb maximum, 39 lb average, to 1 lb minimum.

The crest elevation of the breakwater was determined by considering wave run-up, storm surge, and extreme high tides. Several methods were used to calculate wave run-up that resulted in an average value of 9.5 ft, including storm surge during design storm wave conditions. Using a still water level of 9.6 ft MLLW, a crest elevation of 19 ft MLLW was calculated. Therefore, the new breakwater crest elevation would be 19 ft MLLW. For consistency, the existing breakwater extension crest elevation would be 20 ft MLLW. A crest width of 10 ft was selected based on the armor size and constructability considerations.

The A1 rock would extend down the seaside slope to a 6.5-ft-wide toe configuration at the base of the breakwater. The harbor side A1 rock would extend to a minimum elevation of 0 ft MLLW.

The breakwater extension at the barge landing area was designed with a similar cross-section as that of the main breakwater extension. An excavated toe was used instead of a buttressed toe due to the requirement to maintain adequate width for barge access. Cross sections for these features are shown in figures 19 and 20.

A total of 19,600 CY of A1 rock, 12,900 CY of B1 rock, and 25,900 CY of Core1 rock would be required for construction of the breakwater. A total of 900 CY of A4 rock, 850 CY of B4 rock, and 1,400 CY of Core4 rock would be required for construction of the breach constriction.

Floating Breakwater Design. Methods described in Section 5 of this appendix were used to determine the type and design dimensions for the new concrete floating breakwater. Based on the 50-year design wave height and period for the southwest direction and inner harbor wave criteria, a floating breakwater width of 16 ft and draft of 5 ft was determined. The structure would made-up of segments similar to the existing floating breakwater system currently used at the harbor. A bottom anchor and chain system would be used to hold the segments in position. A total of 732 lineal ft of floating breakwater would be formed by three sections of equal length. A plan and typical section is shown in Figure 21. Final detailed design of the concrete floating breakwater will be performed during preparation of plans and specifications for the construction contract. This will include concrete specifications, floatation design, post-tensioning thru-rod design, anchoring system design, and cathodic protection.

Uplands. No additional uplands would be provided for alternative 1A. Existing uplands area are sufficient for current and future anticipated harbor operations and support. Local interests could expand the existing uplands in the future by excavating into the adjacent slope if necessary. Given the total harbor area of 10 ac, uplands to total harbor area ratio of 0.15 would apply to this alternative. This is significantly less than the required 0.40 ratio; however, an exception to the established criterion is recommended for Port Lions.

Entrance Channel Navigation. The proposed breakwater alignment would create an entrance channel with an effective width of 450 ft at project depth between the breakwater heads. This width exceeds the minimum width needed for the design vessel. It would, however, increase tidal velocities above the existing conditions by pinching off the flow path into and out of the back bay of Settler Cove. Calculations were performed to estimate this increase due to the constriction. It is estimated that the peak velocity at the inflection point of the ebb tide curve would be 0.80 ft-per-second (fps) for the existing condition and 0.96 fps for the Alternative 1A entrance configuration. This velocity increase would not significantly affect navigation. Numerous harbors in the State of Alaska have velocities greater in magnitude. It is anticipated that the design vessel and other vessels in the fleet using the harbor would have little difficulty navigating the new entrance to the harbor even under storm and extreme tide conditions. The existing 150-ft-wide dredged entrance would remain unchanged and continue to be used as it is presently used. It would, however, be considerably less affected by incident wave action from the northeast direction.

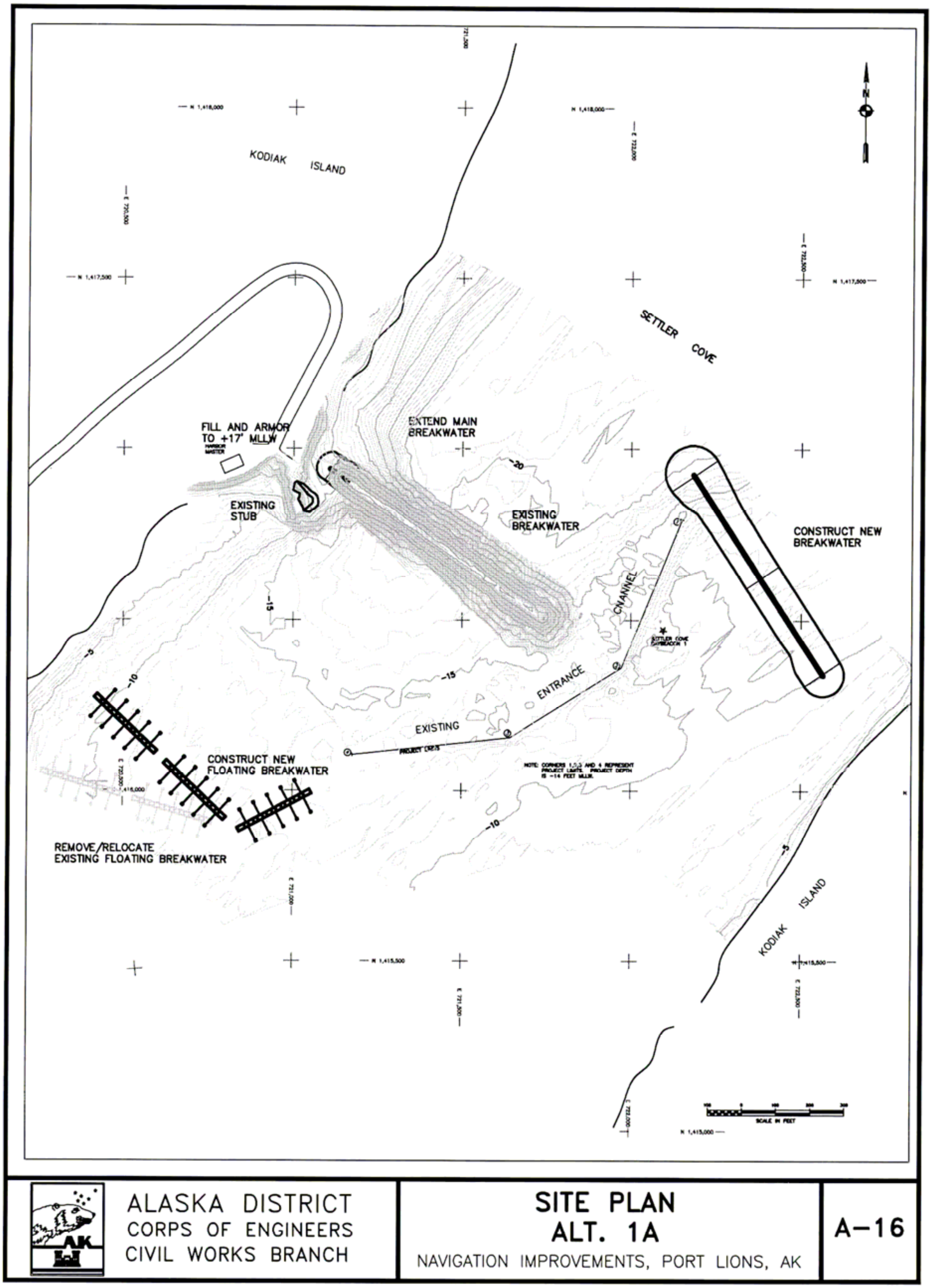

Figure 16. Alternative 1A

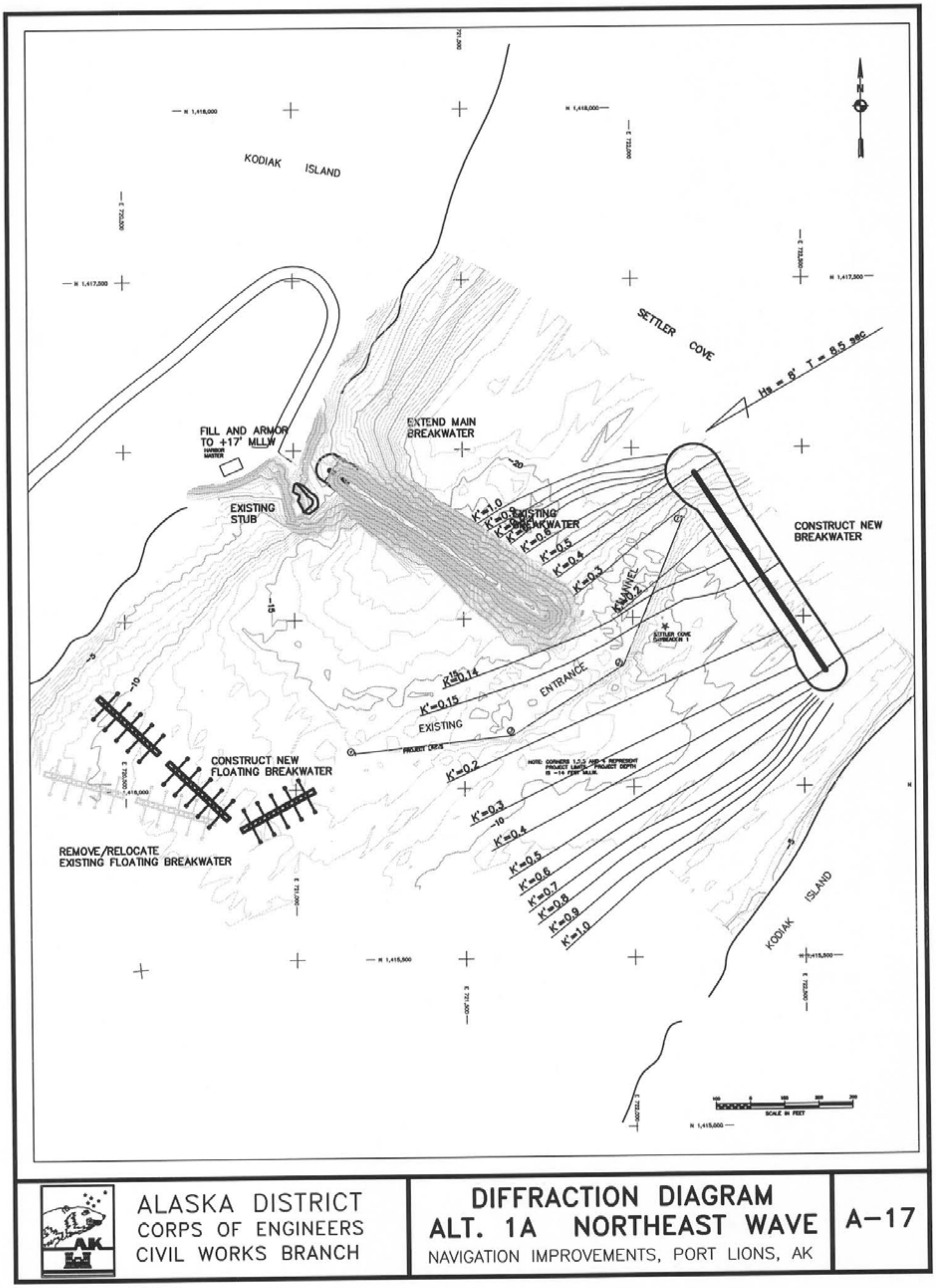

Figure 17. Diffraction Diagram Alternative 1A, NE wave

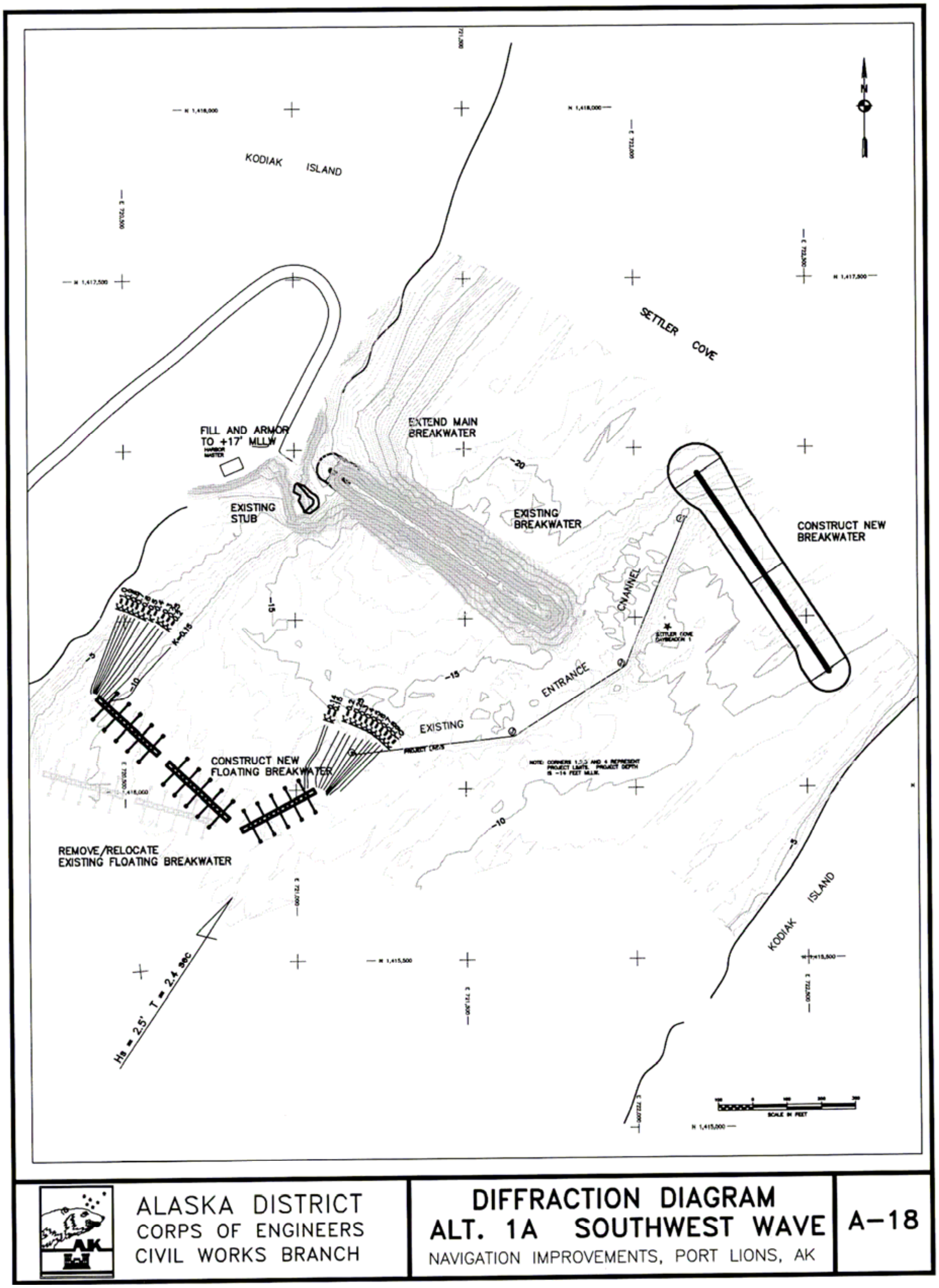

Figure 18. Diffraction Diagram Alternative 1A, SW wave

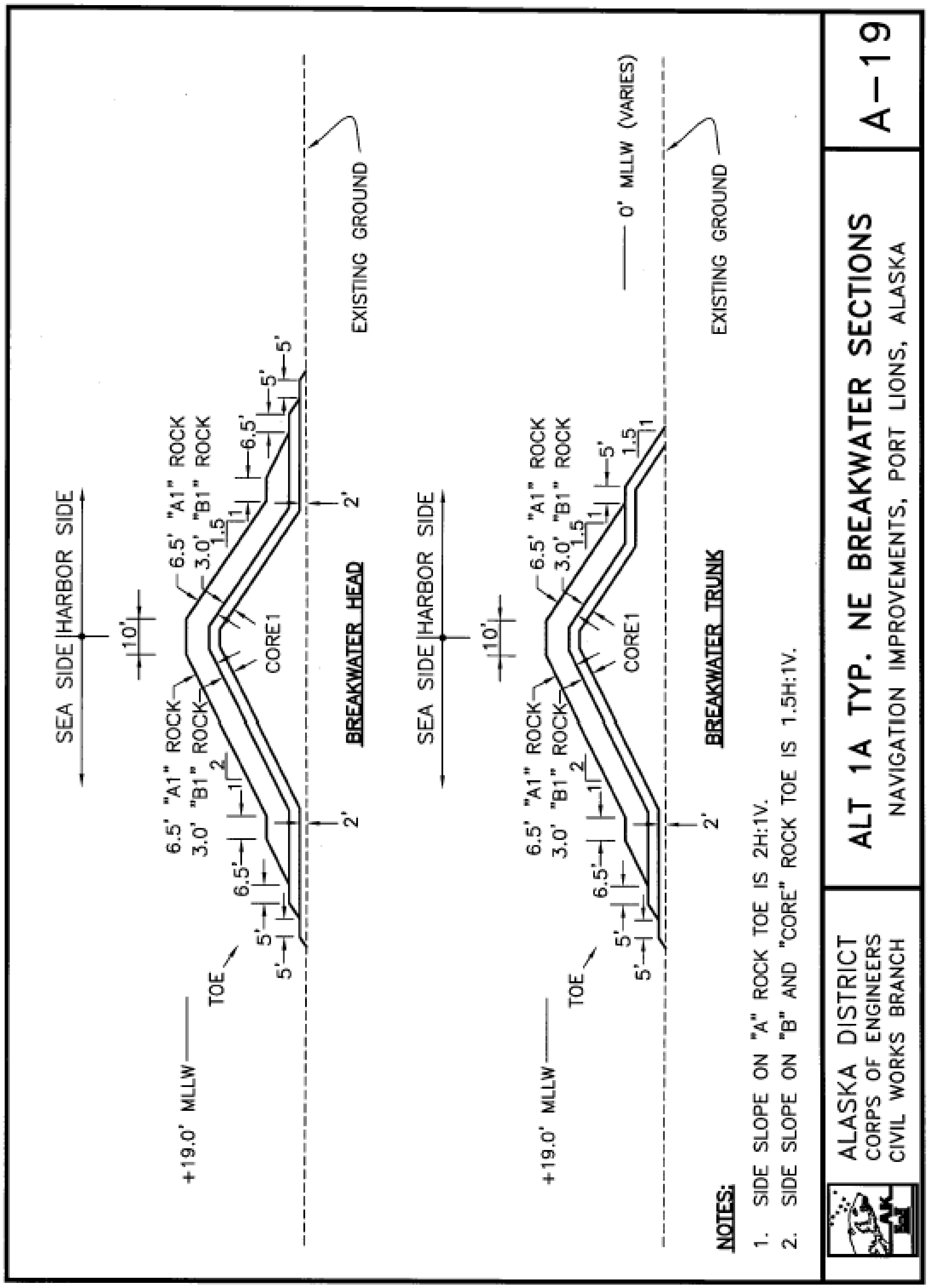

Figure 19. Alternative 1A - Typical NE Breakwater Cross-Sections

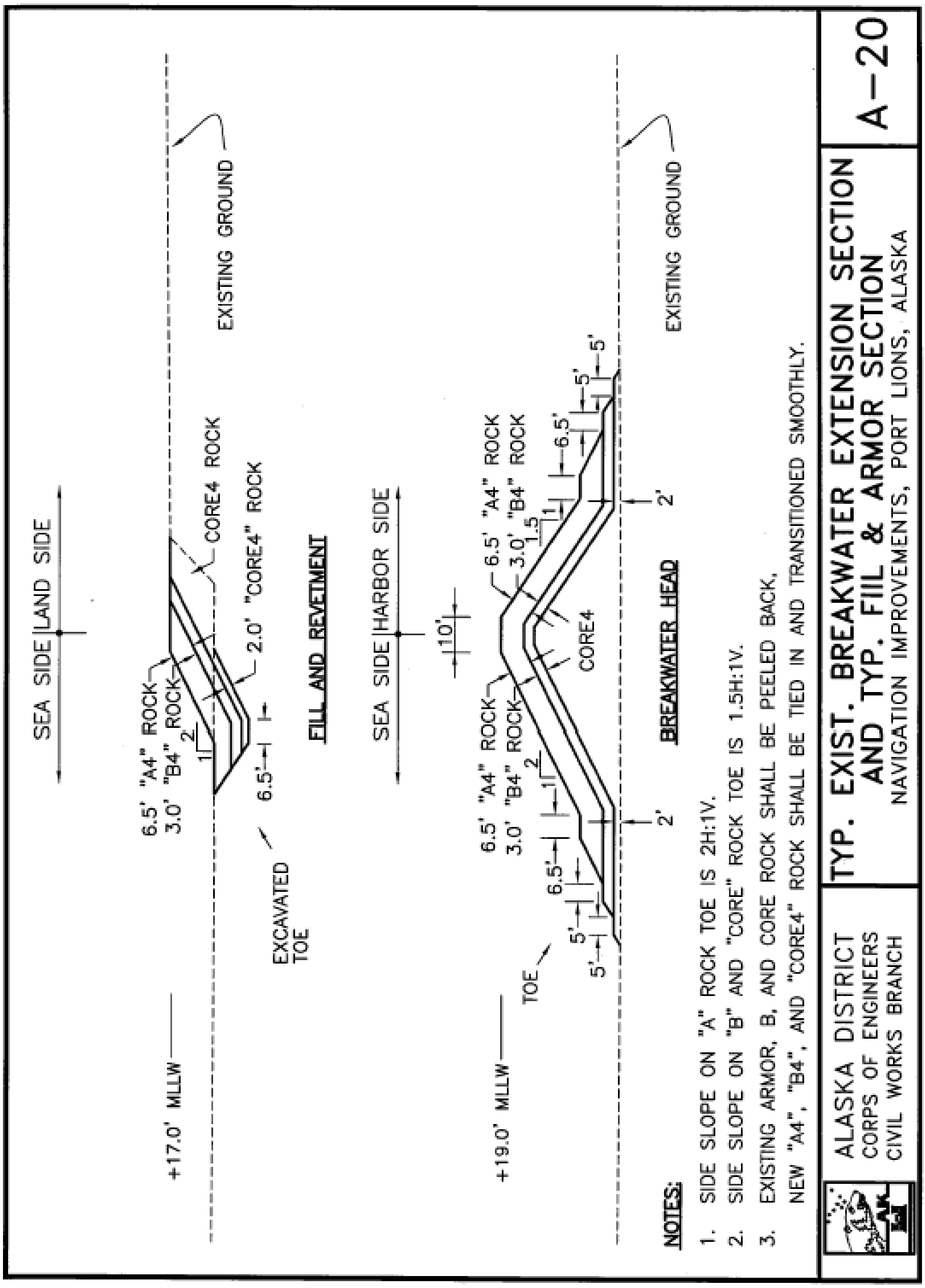

Figure 20. Typical Existing Breakwater Extension Cross-Section

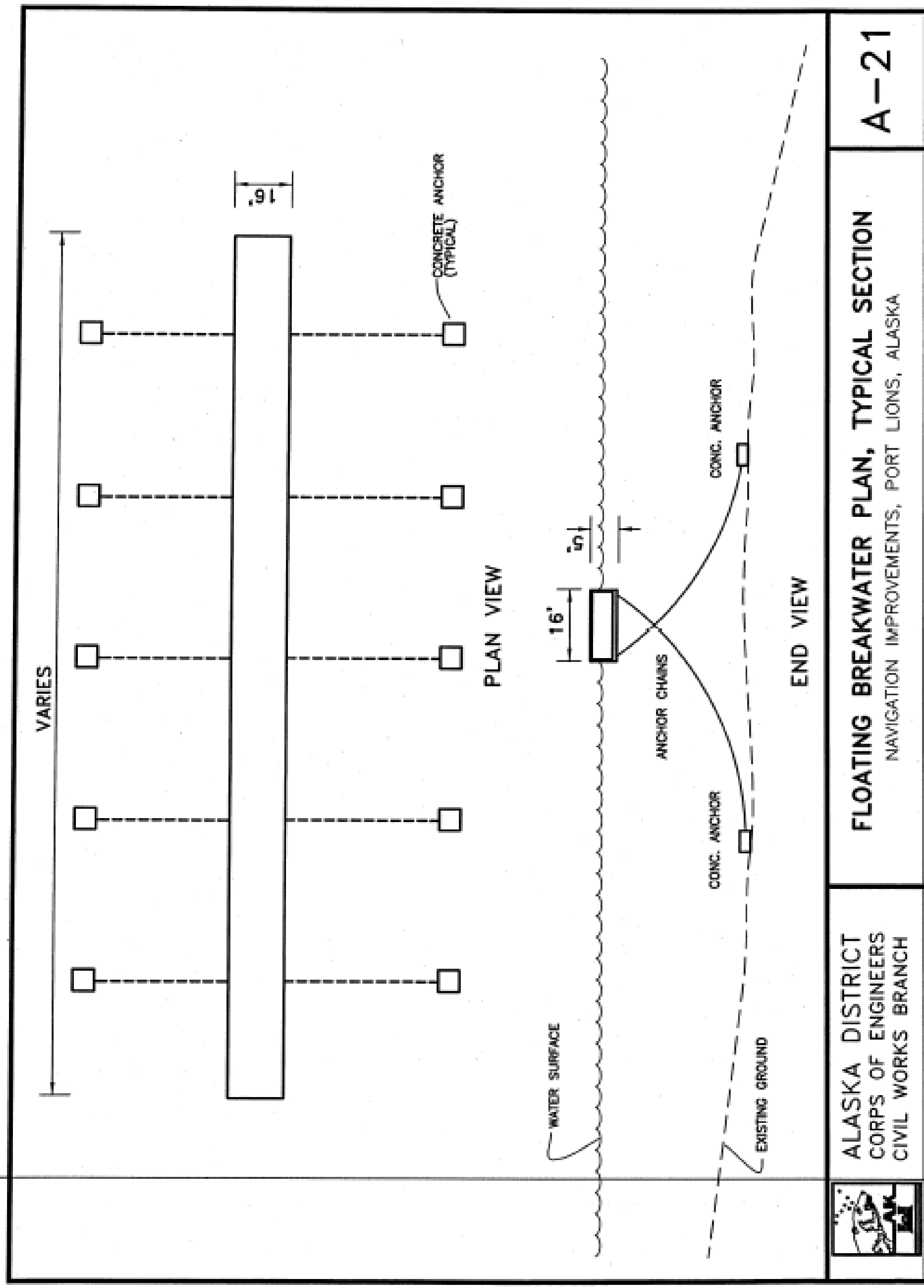

Figure 21. Floating Breakwater Plan, Typical Cross-Section

6.2.2 Alternative 1B.

Alternative 2 is very similar in configuration to alternative 1A. The difference between the two is that the southern limit of the harbor basin would be protected by a rubblemound breakwater instead of a floating breakwater. This alternative, shown in figure 22 incorporates the following: a 700-ft long detached rubblemound breakwater located northeast of the existing breakwater, a 860-ft long detached rubblemound breakwater located southwest of the basin, a 40-ft long extension of the existing breakwater to the west for reduction in the existing breach width, and a 75-ft long extension of the existing stub breakwater at the barge landing to further reduce the breach width. The existing mooring basin would remain unchanged with this alternative. The remaining harbor features would be similar to those of Alternative 1A. An additional new navigation marker light would be established at the head of the new southwest breakwater to guide mariners into the harbor. The new southwest rubblemound breakwater would replace the existing floating breakwaters.

Harbor Basin. The harbor basin would have the same dimensions, depths, and orientation as that for Alternative 1A.

Wave Heights. This alternative would meet the wave criteria established in Section 5 of this appendix along the floats inside the harbor basin. The breakwaters were positioned to reduce incident wave heights from the various directions of exposure to acceptable levels. The maximum wave heights in the mooring areas, based on the 50-year design incident wave, were calculated to be 1 ft and less. Progressively smaller wave heights would occur farther into the harbor mooring area, as shown in the diffraction diagrams in figures 23 and 24. Predicted wave heights inside the harbor under design conditions are calculated by multiplying the incident design wave height by the coefficient (K') indicated. All directions of wave exposure were taken into account in determining the highest wave heights in the mooring area.

Circulation. This alternative would not fully enclose a harbor basin proper, however it would somewhat affect water circulation patterns in the mooring area due to the new southwest rubblemound breakwater. It is estimated that 40 percent of the water in the basin would be exchanged during each tide cycle (tidal prism ratio of 0.4). The aspect ratio of the basin is 1.2. The ratio of the basin planform area (A) to the entrance cross-sectional area (a) is 61. The areas of potentially low exchange were checked to ensure that no more than 5 percent of the total area had exchange coefficients less than 0.15. All parameters meet the harbor design criteria for water quality and circulation.

Shoaling. Shoaling of the entrance channel would not be expected under the same rationale as that for Alternative 1A.

Construction Dredging. No dredging would be required for Alternative 1B.

Maintenance Dredging. Maintenance dredging would be expected to be minimal or not necessary at all in the future under the same rationale as that for Alternative 1A.

Breakwaters. The positioning of the new rubblemound breakwaters would create an entrance channel alignment allowing access from the northeast to the basin similar to Alternative 1A. Maximum depths of water are –13 ft MLLW along the alignment of the southwest breakwater at the head. Foundation materials would be sand, gravel, and rock that would serve as a suitable base for both rubblemound structures.

Rubblemound Breakwater Design. Similar breakwater design methodology described for alternative 1A was used for alternative 1B. Stone size for the southwest rubblemound breakwater was determined using the significant wave height presented in table 4, with a slope of 1.5H:1V on both the harbor and seaside slopes, and a K_d value of 4 for a non-breaking wave. Due to the relatively small design wave height, a two-layer design is proposed for the new southeast breakwater. Armor stone (A1 rock) with a range of sizes from 6,100 lb maximum weight, 4,900 lb average weight, to 3,650 lb minimum weight would be used on the face of the northeast breakwater and breakwater extension. Secondary stone (B1 rock) would range from 3650 lb maximum weight, 490 lb average, to 360 lb minimum weight. Core1 material would range from 360 lb maximum, 49 lb average, to 1 lb minimum. A1 armor stone thickness would be 6.5 ft, and B1 secondary stone thickness would be 3 ft. Armor stone (A3 rock) with a range of sizes from 600 lb maximum weight, 350 lb average weight, to 100 lb minimum weight would be used on the face of the southwest breakwater. Core3 material would range from 100 lb maximum, 4 lb average, to 1 lb minimum. A3armor stone thickness would be 2.5 ft.

The crest elevation of the southwest breakwater was determined by considering wave run-up, storm surge, and extreme high tides. Since the design wave from the southwest direction is relatively small, wave run-up on the breakwater would be minimal. The crest elevation would be at least high enough to remain above water during the extreme high tides. Using an extreme high tide water level of 13 ft MLLW, a crest elevation of 15 ft MLLW was determined. Therefore, the new southwest breakwater crest elevation would be set at 15 ft MLLW. A crest width of 4 ft was selected based on the armor stone size.

The A3 rock would extend down the seaside slope to a 5-ft wide toe configuration at the base of the breakwater. As natural depths vary toward shallower water, the toe elevation would vary as well. The harbor side A3 rock would extend to a minimum elevation of 0 ft MLLW. Cross sections for the southwest rubblemound breakwater are shown in figure 25. A total of 19,600 CY of A1 rock, 12,900 CY of B1 rock, and 25,900 CY of Core1 rock would be required for construction of the northeast breakwater. A total of 900 CY of A4 rock, 850 CY of B4 rock, and 1,400 CY of Core4 rock would be required for construction of the breach constriction. A total of 7,100 CY of A3 rock and 31,400 CY of Core3 rock would be required for construction of the southwest breakwater.

Uplands. Uplands for alternative 1B would be similar to those of Alternative 1A.

Entrance Channel Navigation. The proposed breakwater alignment would have the same navigation considerations as Alternative 1A. Once inside the outer breakwaters, vessels would be able to navigate through the existing dredged entrance channel around the new southwest breakwater head unimpaired since currents and wave action would be minimal.

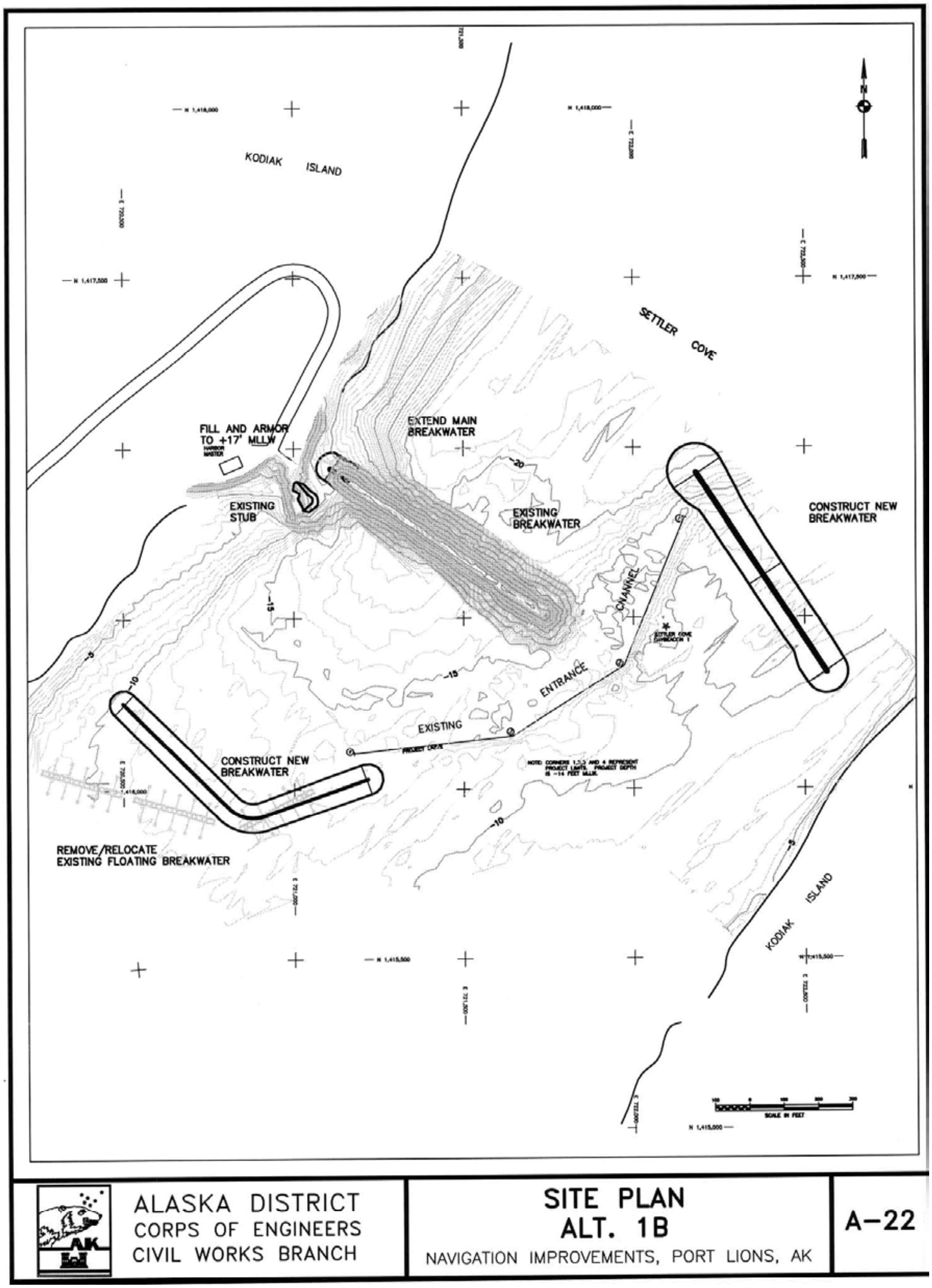

Figure 22. Alternative 1B Plan

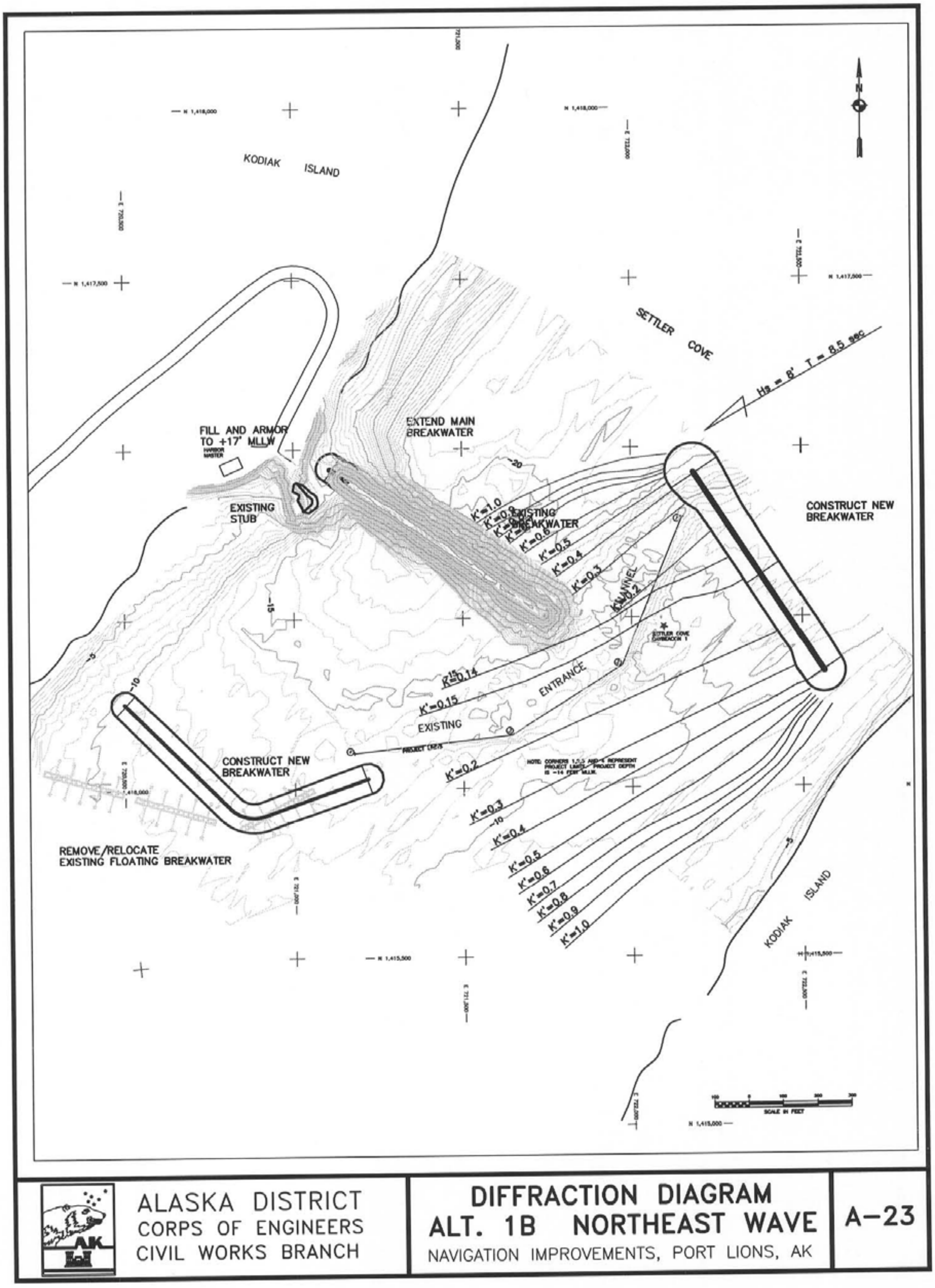

Figure 23. Diffraction Diagram Alternative 1B NE Wave

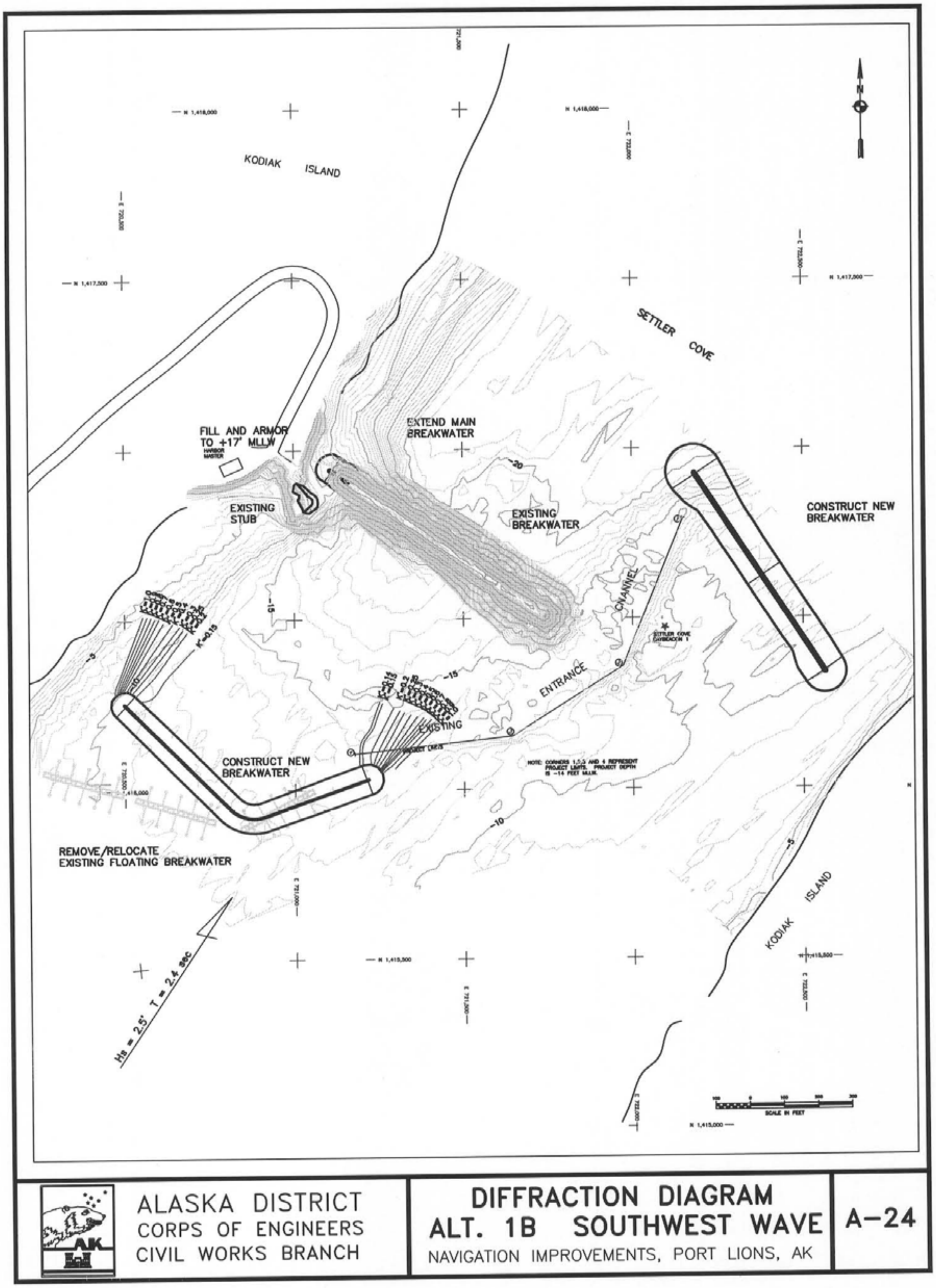

Figure 24. Diffraction Diagram Alternative 1B SW Wave

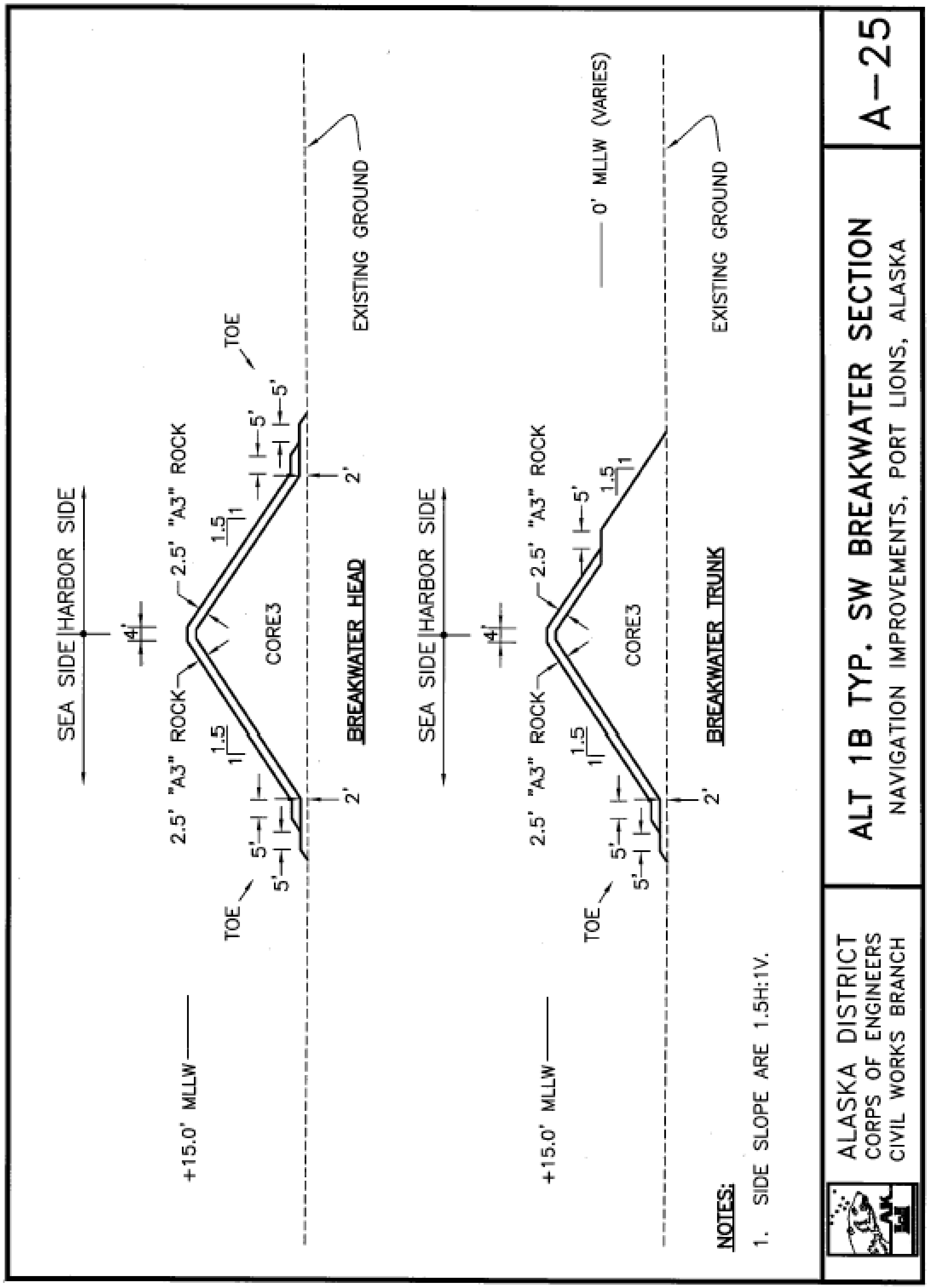

Figure 25. Alternative 1B Typical SW Breakwater Cross-Section

6.2.3 Alternative 3B.

The initial breakwater alignment was conceptualized and provided by the ADOT&PF Ports and Harbors Section. The layout would take advantage of relatively shallower water depths along the eastern perimeter of the basin. Alternative 3B would be protected by a new rubblemound breakwater along its southwestern and eastern perimeter. This alternative, shown in figure 26, incorporates the following: a 1,360–ft long detached rubblemound breakwater located southwest and east of the basin, a 40-ft long extension of the existing breakwater to the west for reduction in the existing breach width, and a 75-ft long extension of the existing stub breakwater at the barge landing to further reduce the breach width. The existing mooring basin would remain unchanged with this alternative. The remaining harbor features would be similar to those of Alternatives 1A and 1B. An additional new navigation marker light would be established at the head of the new southwest breakwater to guide mariners into the harbor. The southwest portion of the new rubblemound breakwater would replace the existing floating breakwaters.

Harbor Basin. The harbor basin would be similar to those of Alternatives 1A and 1B and it would have potential for future expansion. A total combined maneuvering and mooring basin area of approximately 12.0 ac would be available in the basin for alternative 3B.

Wave Heights. Breakwaters were positioned to reduce incident wave heights from the various directions of exposure to acceptable levels. The maximum wave heights in the mooring areas, based on the 50-year design incident wave, were calculated. Progressively smaller wave heights would occur farther into the harbor mooring area, as shown in the diffraction diagrams in figures 27 and 28. Predicted wave heights inside the harbor under design conditions are calculated by multiplying the incident design wave height by the coefficient (K') indicated. Both directions of wave exposure were taken into account in determining the highest wave heights in the mooring area. For waves from the northeast direction, SPM methods were used to calculate diffraction coefficients: wave diffraction around the existing breakwater head followed by diffraction around the proposed new breakwater head. K' values are shown on figure 27. Diffracted wave heights inside the harbor would fall within the criteria established for the project (ie: waves less than one ft in height within the mooring area during design storm conditions).

Circulation. This alternative would more fully enclose a harbor basin proper. It is estimated that 46 percent of the water in the basin would be exchanged during each tide cycle (tidal prism ratio of 0.46). The aspect ratio of the basin is 1.2. The ratio of the basin planform area (A) to the entrance cross-sectional area (a) is 280. The areas of potentially low exchange were checked to ensure that no more than 5 percent of the total area had exchange coefficients less than 0.15. All parameters meet the harbor design criteria for water quality and circulation.

Shoaling. Shoaling of the entrance channel would not be expected since there is little evidence of significant long-shore transport of sediments at the site. There has been no shoaling of the existing entrance channel since initial dredging. The construction of the new breakwater would not alter the existing sedimentation patterns in the project vicinity.

Construction Dredging. No dredging would be required for Alternative 3B.

Maintenance Dredging. Maintenance dredging would be expected to be minimal or not necessary at all in the future under the same rationale as that for Alternatives 1A and 1B.

Breakwaters. The positioning of the new rubblemound breakwater would create an entrance channel alignment allowing access from the northeast to the basin around the heads of the existing breakwater and the new breakwater. Existing depths would remain unchanged however the channel width would be significantly reduced. Maximum water depth at the head of the new breakwater is –15 ft MLLW. Foundation materials would be sand, gravel, and rock that would serve as a suitable base for the new rubblemound structure. The positioning of the new breakwater would create more of a 90-degree turn into the harbor from the northeast.

Rubblemound Breakwater Design. Similar breakwater design methodology described for alternatives 1A and 1B was used for alternative 3B. The breakwater was divided into three segments representing varying degrees of wave exposure. Stone size for the southwest rubblemound breakwater segments was determined using the significant wave heights of 7.2, 5.2, and 2.5 ft for the north, middle, and south segments respectively. The slope was 1.5H:1V on both the seaside and harbor side for the northerly (most exposed) segment and the southwesterly segments with a K_d value of 4 for a non-breaking wave. Armor stone (A1 rock) with a range of sizes from 6,100 lb maximum weight, 4,900 lb average weight, to 3,650 lb minimum weight would be used on the face of the northerly segment of the breakwater. Secondary stone (B1 rock) would range from 3,650 lb maximum weight, 490 lb average, to 360 lb minimum weight. Core1 material would range from 360 lb maximum, 49 lb average, to 1 lb minimum. A1 armor stone thickness would be 6.5 ft, and B1 secondary stone thickness would be 3 ft. Armor stone (A2 rock) with a range of sizes from 2,500 lb maximum weight, 2,000 lb average weight, to 1500 lb minimum weight would be used on the face of the breakwater middle segment. Secondary stone (B2 rock) would range from 1,500 lb maximum weight, 200 lb average, to 150 lb minimum weight. Core2 material would range from 150 lb maximum, 20 lb average, to 1 lb minimum. A2 armor stone thickness would be 4.5 ft, and B2 secondary stone thickness would be 2 ft. Armor stone (A3 rock) with a range of sizes from 600 lb maximum weight, 350 lb average weight, to 100 lb minimum weight would be used on the face of the southerly segment of the southwest breakwater. Core3 material would range from 100 lb maximum, 4 lb average, to 1 lb minimum. A3 armor stone thickness would be 2.5 ft. Cross sections for the rubblemound breakwater segments are shown in figures 29 and 30.

The crest elevation of the southwest breakwater was determined by considering wave run-up, storm surge, and extreme high tides. Since the design wave from the southwest direction is relatively small, wave run-up on the breakwater would be minimal. The crest elevation would, however, need to be at least high enough to remain above water during the extreme high tides. Using an extreme high tide water level of 13 ft MLLW, crest elevations of 19 ft MLLW for the northerly and middle segments and 15 ft MLLW for the southerly segment were determined. Therefore, the new southwest breakwater segments crest elevations would be set at 19 and 15 ft MLLW, respectively. A crest width of 10 ft for the northerly segment, 7 ft for the middle segment, and 4 ft for the southerly segment was selected based on the armor stone size.

The A1, A2, and A3 rock would extend down the seaside slope to 6.5 and 5-ft wide toe configurations respectively at the base of the breakwater segments. As natural depths vary toward shallower water, the toe elevation would vary as well. The harbor side A1, A2, and A3 rock would extend to a minimum elevation of 0 ft MLLW.

A total of 17,100 CY of A1 rock, 7,700 CY of B1 rock, and 11,500 CY of Core1 rock would be required for construction of the northerly segment of the southwest breakwater. A total of 900 CY of A4 rock, 850 CY of B4 rock, and 1,400 CY of Core4 rock would be required for construction of the breach constriction. A total of 9,800 CY of A2 rock, 6,400 CY of B2 rock, and 23,500 CY of Core2 rock would be required for construction of the middle segment of the southwest breakwater. A total of 3,200 CY of A3 rock, and 13,800 CY of Core3 rock would be required for construction of the southerly segment of the southwest breakwater.

Uplands. Uplands for alternative 3B would be similar to those of Alternatives 1A and 1B.

Entrance Channel Navigation. The proposed breakwater alignment would create an entrance channel with an effective width of 100 ft at project depth between the breakwater heads. This width was established based on the design vessel for the project. Vessels entering the harbor would use the existing dredged entrance channel outside and around the head of the existing main breakwater, then make a 90-degree turn to starboard to enter the harbor through the new 100-ft wide entrance between the new and existing breakwaters. Alternative 3B would alter the alignment and reduce the length and effective width of the southern one-third of the existing entrance channel. This represents a significant constriction compared to the existing entrance, however it meets minimum criteria for the design vessel and would be fully functional. It would be anticipated that local harbor users would immediately become familiar with the new configuration and adjust their entrance and exit maneuvers for safe navigation. Tidal currents in the new entrance channel would be anticipated to be minimal and not a significant concern with respect to navigability of the new harbor entrance.

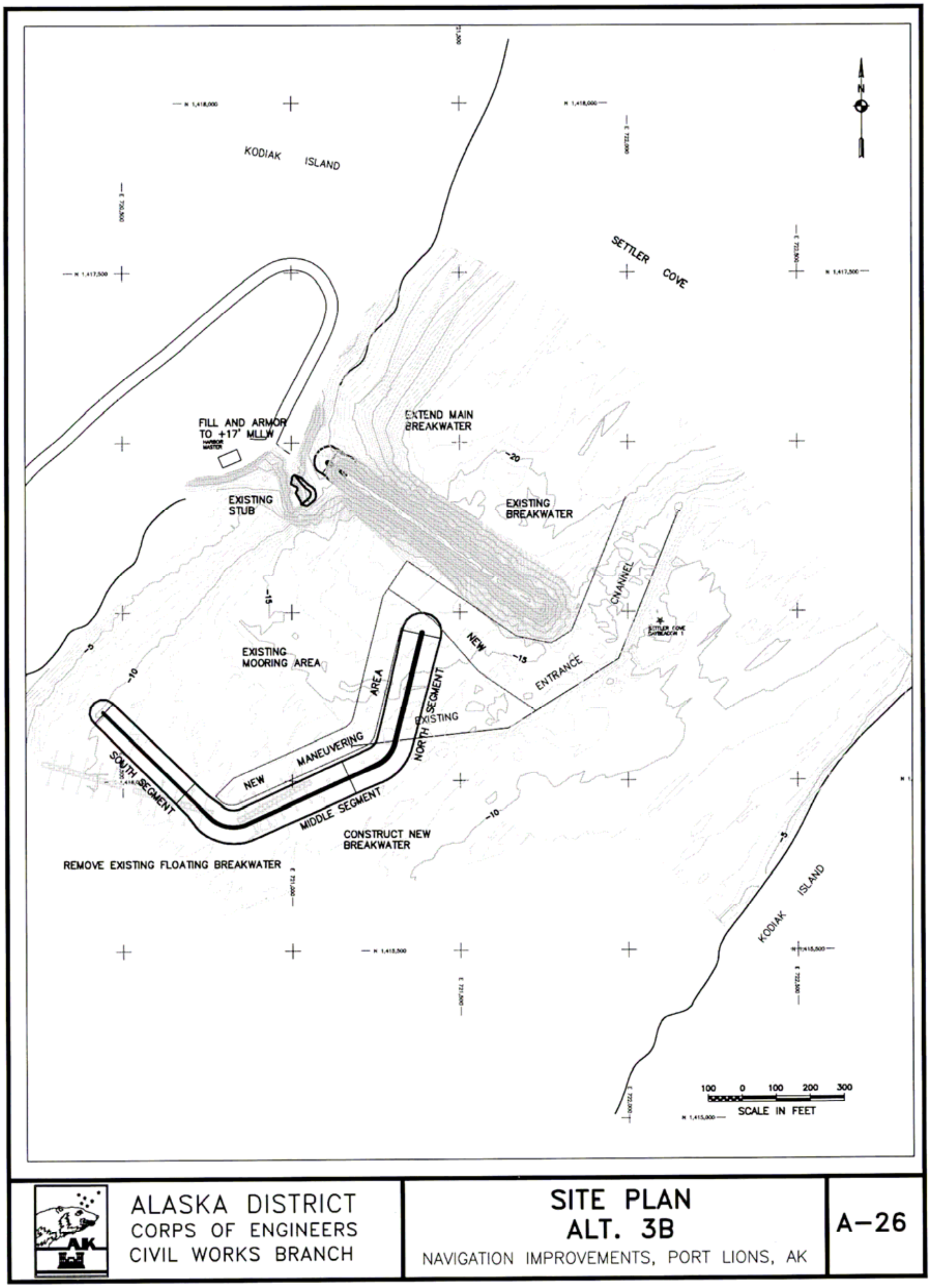

Figure 26. Alternative 3B Plan

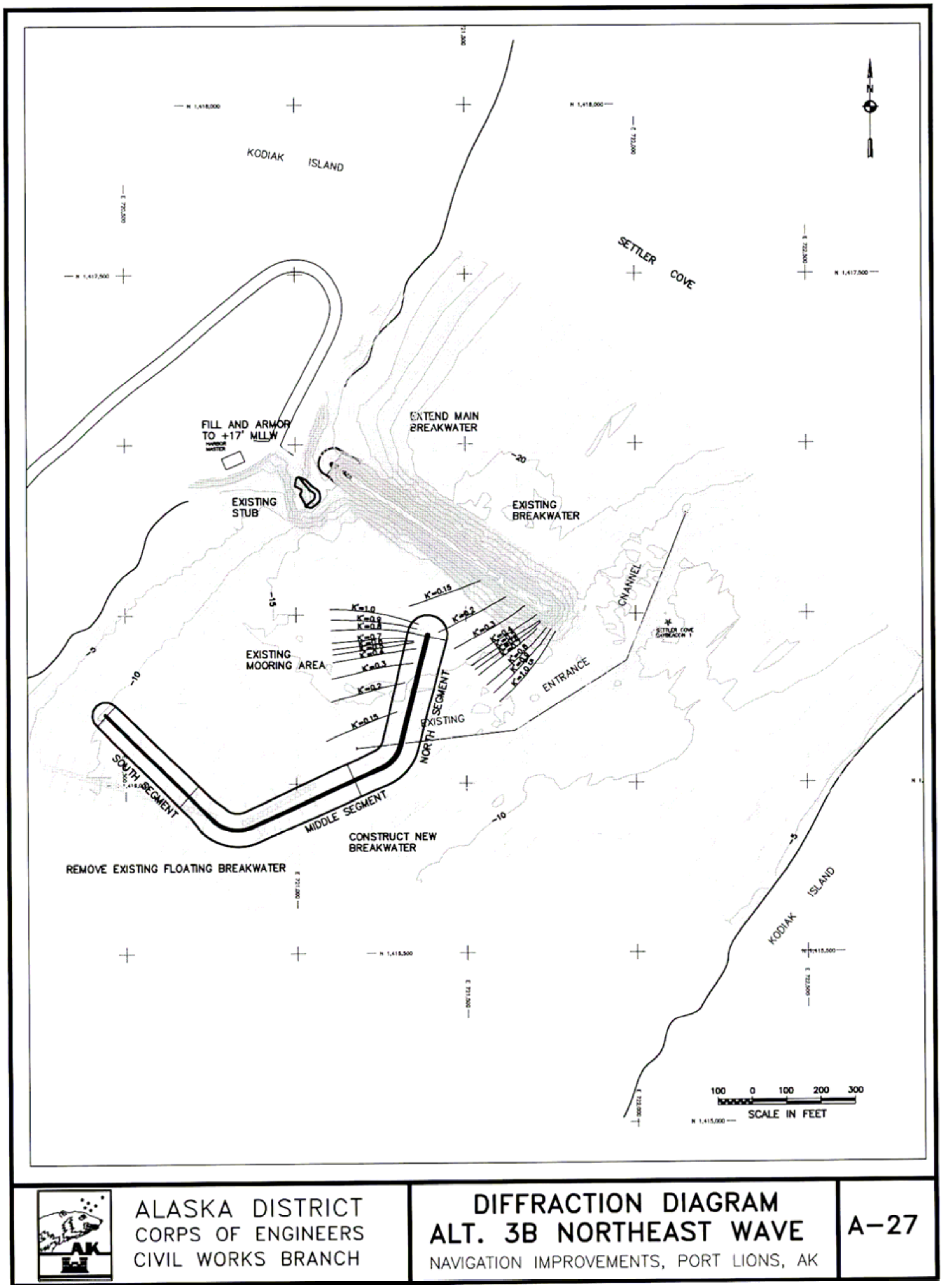

Figure 27. Diffraction Diagram Alternative 3B NE Wave

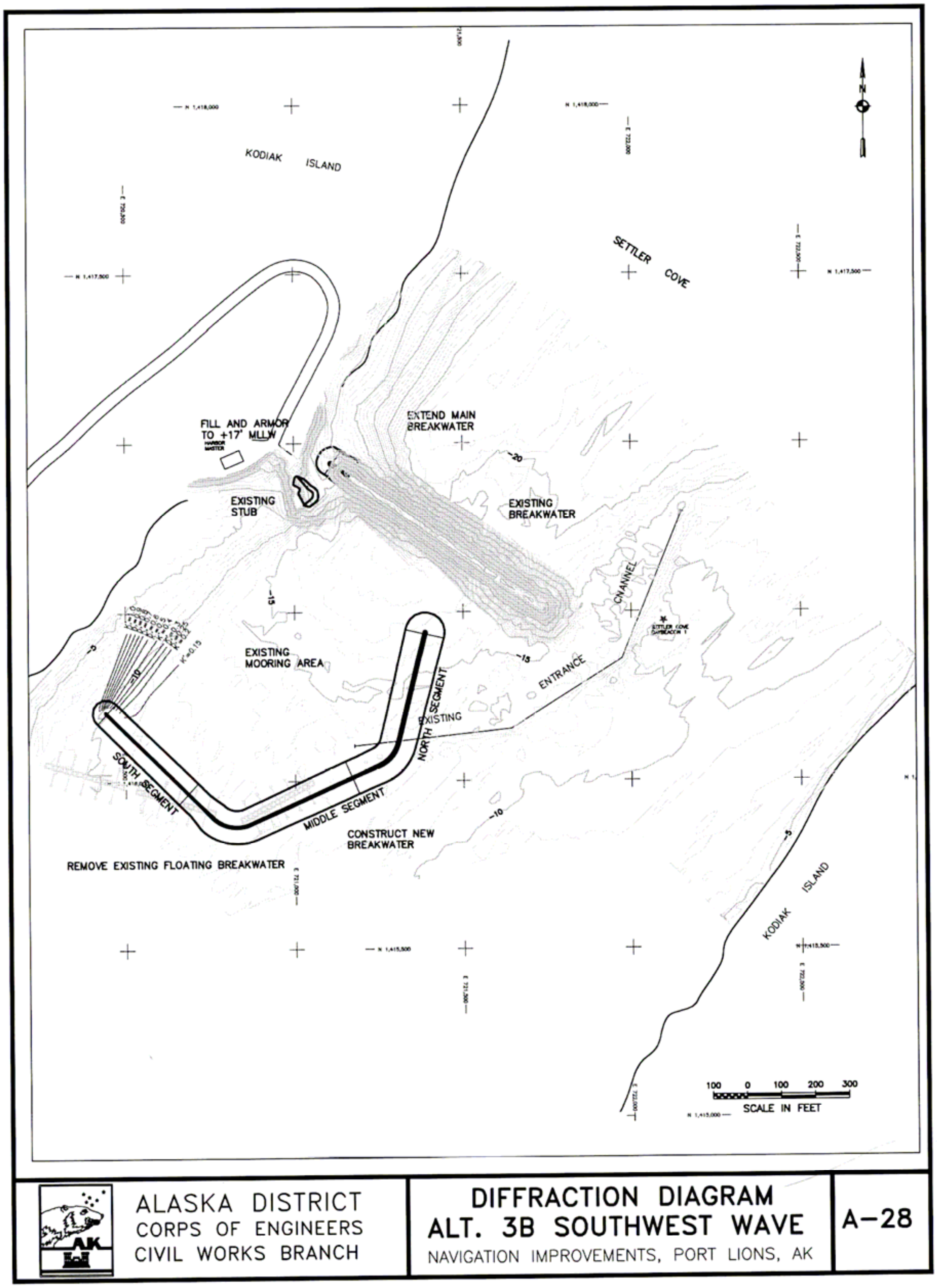

Figure 28. Diffraction Diagram Alternative 3B SW Wave

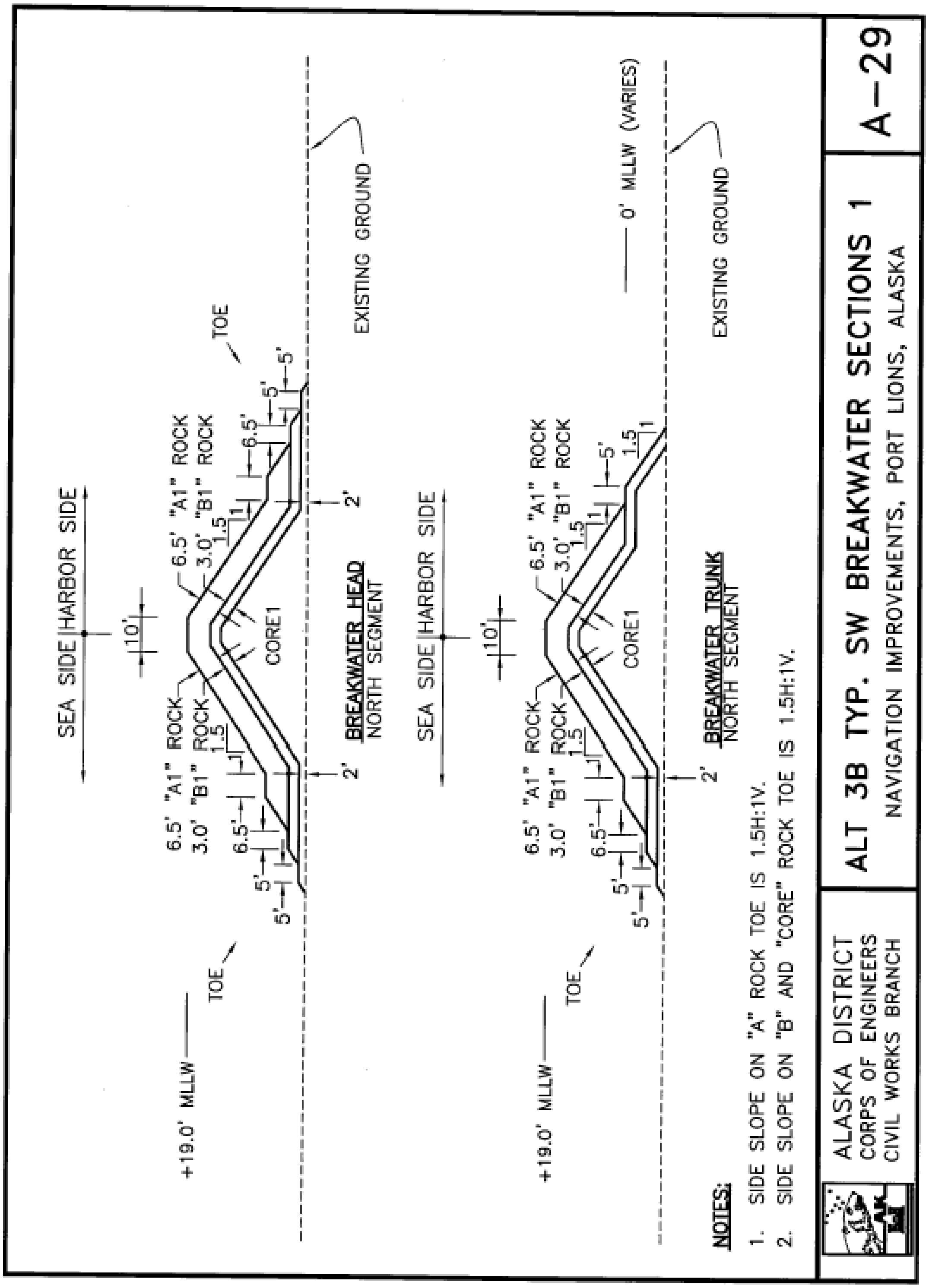

Figure 29. Typical Breakwater Cross-Sections 1

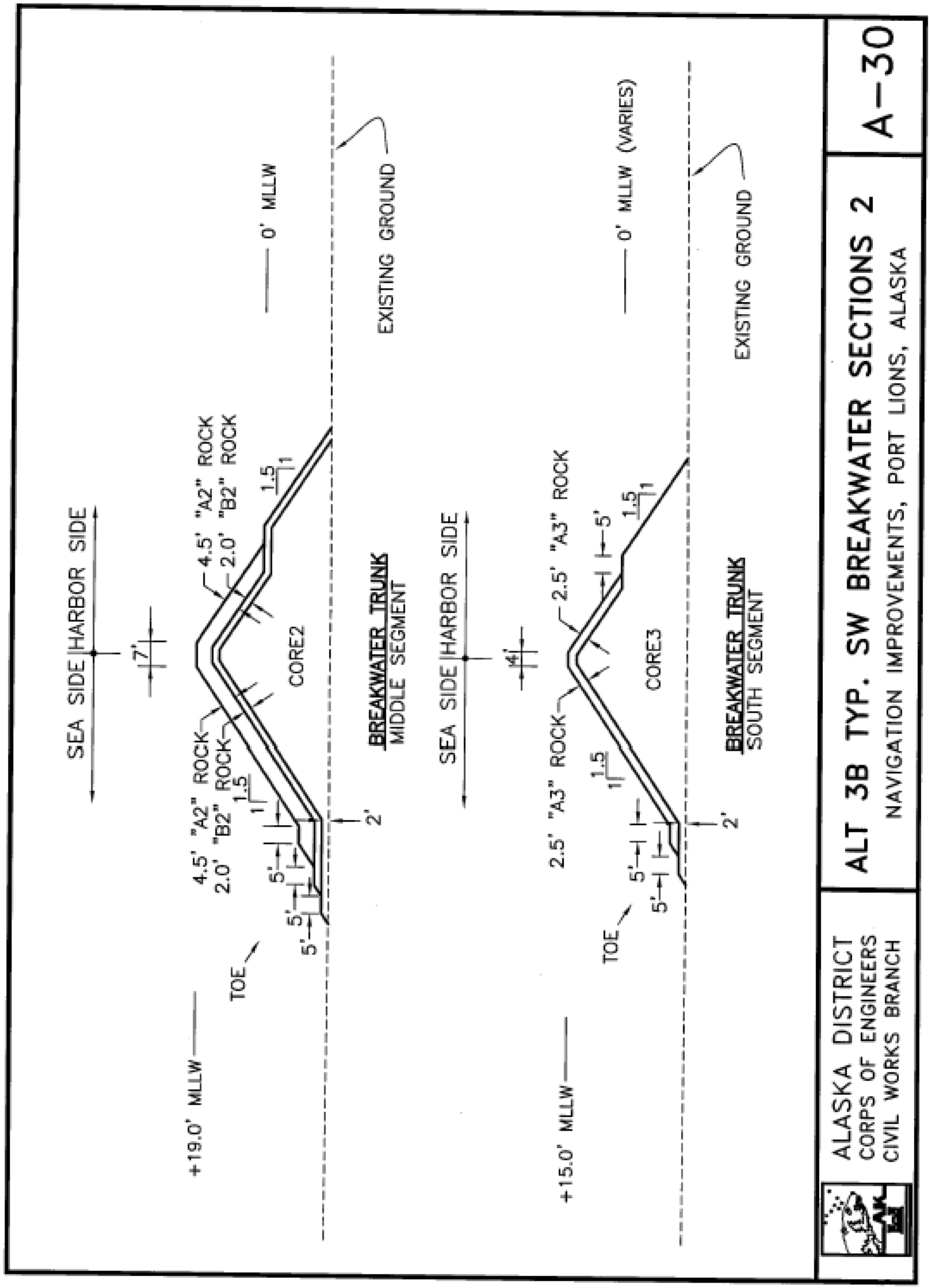

Figure 30. Typical Breakwater Cross-Sections 2

7.0 PLAN IMPLEMENTATION

7.1 Aids To Navigation

As part of the construction of the project, navigation marker bases would be constructed at the head of the proposed breakwaters to define the entrance to the harbor. Correspondence with the U.S. Coast Guard has been conducted to assure that necessary marking of the entrance channel was considered. A new navigation light would be incorporated into the head of the new breakwaters for any of the alternatives. The new floating breakwater would be retrofitted with a navigation marker base as well. The Coast Guard would install the navigation lights and signage after construction is completed.

The existing day marker pile outside of the existing breakwater would remain. However, it would be pulled and reset since it is currently leaning substantially and may be unstable.

7.2 Operation and Maintenance Plan

Operation of the existing mooring area portion of the project would remain the city of Port Lions' and State of Alaska's responsibility. The Federal Government would be responsible for the breakwaters and the entrance channel portions of the project. The Alaska District, Corps of Engineers, would visit the site periodically to inspect the breakwaters and perform hydrographic surveys at 3- to 5-year intervals for the dredged areas. The surveys would be used to verify whether the maintenance dredging is warranted. Maintenance requirements for the breakwaters would be determined from the surveys and inspections. Local and Federal dredging requirements, if necessary, would probably be combined, so there would be only a single mobilization and demobilization cost.

Minimal maintenance dredging is anticipated with any of the alternatives. It is estimated that essentially no maintenance dredging in the entrance channel and mooring basin would be necessary over the remaining 30-year project life. Additionally, minimal maintenance would be anticipated over another 20 years beyond that. The existing project has shown that it is essentially maintenance free with respect to the entrance channel and mooring area depths.

For the breach, the difference between alternatives 1b and 3b will be small. The shoaling in the breach will be similar for both alternatives. The only change that would be anticipated would be to the sediments that reside in the channel off the end of the breakwater. Plan 1b will nearly double the tidal velocities around the breakwater heads so that the fine sediments that are transported by tidal currents will migrate to an area of lower energy.

The largest effect on the sediments will be from the breach alternatives. The lengthening of the existing breakwater shoreward will dissipate the wave energy within the harbor and the breach could shoal in faster than the existing configuration. Although the rate of transport would be similar for each alternative the narrower breach provides a smaller reservoir for storage. Much of the rate of shoaling will depend on whether the community continues to use this deposit as a material source and whether or not the volume of material continues to decline. Also, the source of sediments may be diminished since it is estimated that much of this material was dredged material disposed of on the seaward side of the breakwater when the entrance channel was dredged back in the early 1980's.

The existing breakwater (after its repair in the early 1980 's) and the new breakwaters were designed to be stable for the 50-year predicted wave conditions. No significant loss of stone from the rubblemound structure is expected over the life of the project. It is estimated that at the worst case, 3 percent of the armor stone would be replaced every 15 years. Since stone quality would be strictly specified in the contract, little to no armor stone degradation is anticipated. The armor stone on the existing breakwater was inspected in 2002 and found to be in excellent condition with no visible signs of degradation, fracture, or slaking.

Maintenance of the floating breakwater would be a Federal responsibility unless the harbor users intend to use it as a mooring float or dock for transient vessels. Condition of the concrete, flotation, connections, anchoring system, and cathodic protection would be evaluated and maintenance requirements would be determined by periodic inspections. It is estimated that approximately 5 percent of the connections and 2 percent of the concrete deck area would require repairs at 15-year intervals based on past performance of similar structures around the state assuming it is not used for moorage.

During the winter months, icing conditions could occur in the mooring and maneuvering areas of any of the alternatives. Possible measures to address such conditions include periodically running a vessel in the affected areas to break up the ice enough to allow continued navigation and mooring. This would be the responsibility of local interests.

7.3 Detailed Quantity Estimates

Detailed estimates of quantities for Federal features (breakwaters) for all five alternatives were performed for this appendix. Quantities for local features were also estimated. Quantity estimates were based on hydrographic surveys performed in 2002 by contract to the Corps of Engineers. The AutoCAD and Land Development software, as well as Excel spreadsheet quantity calculation programs were used to determine the quantities. The quantities were checked and verified to be within 10 percent by the ADOT&PF using independent methods.

Final bid items and quantities will be determined during preparation of plans and specifications for the construction of the project. These will be verified and checked through an independent technical review process prior to advertisements for bid.

7.4 Construction Schedule

Major construction items include the rubblemound breakwaters, floating breakwaters, and reconstruction or replacement of the float system. The rubblemound breakwaters would likely be constructed first. The floating breakwater segments for alternatives that incorporate these structures would likely be constructed in Tacoma, Washington and barged to the site for assembly and positioning. The estimated construction time is 24 months. Construction scheduling should minimize conflict with the continued use of the existing harbor facilities. The existing harbor would remain operational during construction. Project specifications would detail environmental and harbor operation time restrictions for the contractor not to conduct certain activities. The inner harbor facilities such as the float system and docks etc. would be constructed by separate contract after the Federal project was completed and would be the responsibility of the local sponsor to fund, design, construct, and maintain.

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